Estimating lateral support pressure for rock cuts at Washington-Dulles International Airport expansion

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ABSTRACT: Washington-Dulles International Airport (Virginia, USA) has been expanding its capacity since 1999. This expansion has required extensive vertical, open-cut rock excavations to depths of up to approximately 20 m adjacent to existing infrastructure for construction of new below-ground stations for the new Automated People Mover (APM) passenger transportation system. As the construction could not impact the airport operations, the selection of support pressures for the rock excavations had to balance the projects' risks and construction costs. Site investigation identified that bedding dipping at approximately 30° would be the primary hazard to the vertical rock cuts. Extensive laboratory testing and field observations suggested that the potential for a large slide along a bedding plane was relatively low. Therefore, design lateral pressures for permanent station walls were based on a potential failure model of local wedge/block failure. The reduction in lateral support requirements for the permanent below-grade walls for the APM station structures resulted in significant cost savings to the project. Detailed field mapping combined with automated instrumentation were successfully used to confirm the design assumptions.

1 INTRODUCTION

Estimating the support pressure for a rock cut depends on the assumed failure mode and that failure mode is a function of the rock mass and its geological history and the method of excavation. At one extreme the rock mass can be a homogeneous intact block whereby the designer is only concerned with elastic deformations and for such situations no support is required. At the other end of the spectrum the rock mass may be so heavily jointed that the rock mass behaves as an assembly of discrete blocks. For this situation the failure mode may be described as an active wedge and the support must be designed to stabilize this wedge. Most rock masses lie between these two end members, and the support requirements can vary significantly.

Sedimentary rocks are characterized by transversely isotropic layers. Such layers may end up with a weak plane separating the beds and these planes can occur as thin mm-scale partings that can extend for hundreds of metres (Fookes 1997). The origin of these bedding planes varies, i.e. depositional and/or tectonic, but in most cases a bedding plane is formed that can be significantly weaker than the adjacent rock. When attempting a rock cut in a sedimentary sequence that is dipping into the excavation the designer must decide if a bedding plane exists, the extent of this bedding plane, and its strength, in order to determine the support pressure. The calculations are straightforward and the challenge lies in assessing the strength and continuity of the bedding planes. For such situations traditional site investigations are seldom adequate as detection of these bedding-planes can be easily missed. This was the challenge for the Washington-Dulles International Airport (Dulles Airport) expansion project (Figure 1).

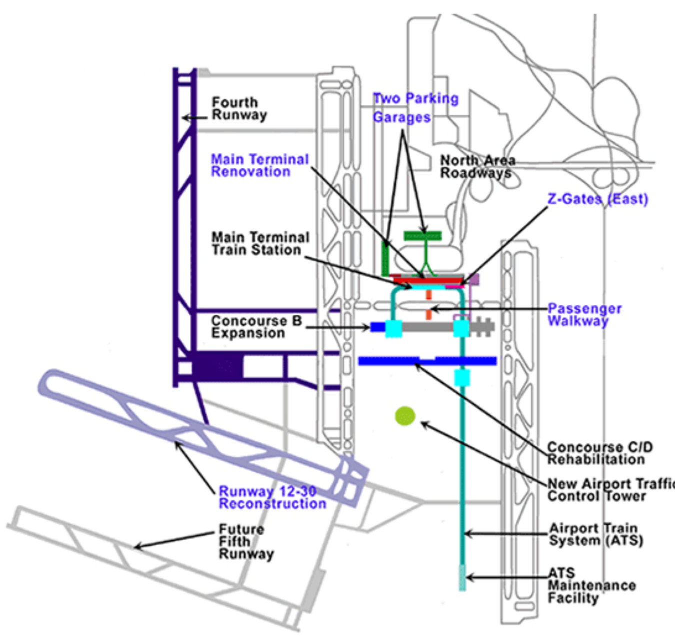

Figure 1: Major projects map for Washington-Dulles International Airport, www.mwaa.com/dulles/d2_dulles_development_2/d2_home.

Dulles Airport (Virginia, USA) has been expanding its capacity since 1999. The present stage of expansion is called the D2 program. This expansion includes extensive vertical, open-cut rock excavations to depths of up to approximately 20 m adjacent to existing infrastructure to construct below-grade stations for the new Automated People Mover (APM) passenger transportation system. As the construction could not impact the airport operations, the selection of support pressures for the rock excavations had to balance the projects' risks and construction costs.

The observational method describes a risk-based approach to geoengineering that employs adaptive management, including advanced monitoring and measurement techniques, to substantially reduce costs while protecting capital investment, human health, and the environment (NRC 2006). Development of the observational method in geoengineering is generally attributed to Terzaghi (Peck, 1969). The essential elements of the observational method that were implemented at Dulles Airport to establish the lateral pressures required to support the rock cuts can be summarized as follows: Establish the geological model for the site; Determine the likely modes of failure and the appropriate method of calculation; Specify a construction procedure consistent with the geological model and the possible mode(s) of failure; Instrument and monitor the excavation sequence and slope performance; Compare predicted and measured parameters to ensure the design assumptions are valid; Change the design as needed.

In this paper we describe the geotechnical investigations, the design sequence, the recommended support pressures, and the results from constructing several cuts.

2 GEOLOGICAL/GEOTECHNICAL SETTING

2.1 *Geological model*

Hoek and Bray (1977), in their book on Rock Slope Engineering say, in the context of regional geological investigations, "A frequent mistake in rock engineering is to start an investigation with a detailed examination of drill cores. While these cores provide essential information, it is necessary to see this information in the context of the overall geological environment and it is therefore useful to start an investigation by building up a picture of the regional geology." This approach was used for characterization of the rock mass, and development of the geological model for design support pressures for deep excavations at the Dulles Airport.

The project site lies within the Culpeper Basin, a geologic feature that extends from South Central Virginia to western Maryland. The predominant rocks are mudstones, siltstones, shales, and sandstones of the Culpeper Group of Triassic age (Toewe, 1966). The regional joints in these Triassic "redbeds" are subvertical and strike North-South and East-West (Toewe, 1966, Nickelson & Hough, 1967, Lee, K.Y. & A.J. Froelich 1989). While the region has been uplifted and intruded by igneous rocks, the sedimentary sequence is relatively undeformed regionally. Geotechnical investigations confirmed that at the Dulles Airport site the structural features that could impact the design of the rock cuts were the bedding planes and the regional subvertical fractures. The bedding planes were the dominant feature encountered and strike approximately North-South and dip towards the West, while the subvertical fractures were often non-persistent. The non-persistent nature of these fractures is consistent with the regional deformation history experienced by these rocks (Price and Cosgrove, 1990).

2.2 *Geotechnical model*

Geotechnical investigations pertaining to the rock excavations for a variety of Dulles Airport expansion projects, including open-cut, mined and bored tunnels, new concourses, existing concourse expansion, and below-ground APM stations have been ongoing since 1999. The field investigations have consisted of traditional and oriented cored boreholes, pressuremeter testing, seismic refraction surveys, optic and acoustic televiewer surveys, and groundwater studies. Samples from cored boreholes were tested to determine the strength and deformation characteristics, as well the durability of the rock.

Because of the sedimentary layering it is challenging to obtain good core recovery and undisturbed samples. During the most recent rock mass characterization study for the new Tier 2 (Concourse C/D) and West station project in 2006, the average core recovery was 98% and the average RQD was 72%. From all the cored holes the majority of the RQD values were >90% (Figure 1).

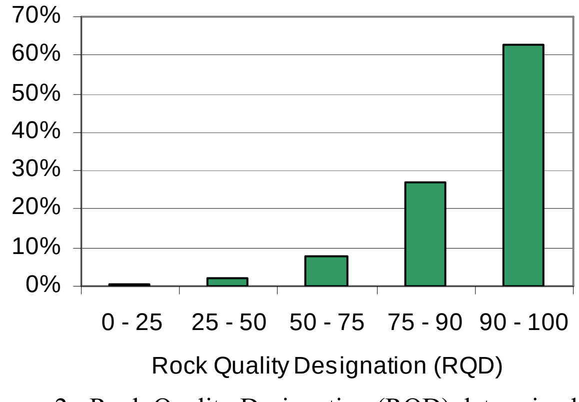

Figure 2: Rock Quality Designation (RQD) determined from various boreholes from the site investigations for the Dulles Airport rock excavations.

The investigations concluded that the discontinuities along bedding planes posed the biggest threat to the stability of the planned excavations, as they dipped between 20 to 30° towards the west. While steeply dipping fractures were observed, they often appeared as rough to undulating in the cores and did not occur as well defined joint sets. Figure 3 shows an equal angle lower hemisphere stereonet of the bedding plane and joint orientations determined from the early site investigation work (Haley & Aldrich, 2001). Based on these investigations it was decided that an average bedding plane dip of 30° would be used for the design of the open cuts. It is clear from Figure 3 that the orientation of the joint sets is less well-defined but given the steep dip of these features the vertical and subvertical boreholes may have biased those results. Recent Acoustic and Optical Televiewer surveys in vertical coreholes confirmed the earlier conclusion of the distinct bedding plane dip and the discontinuous nature of the steeply dipping joints (Schnabel Engineering, 2006).

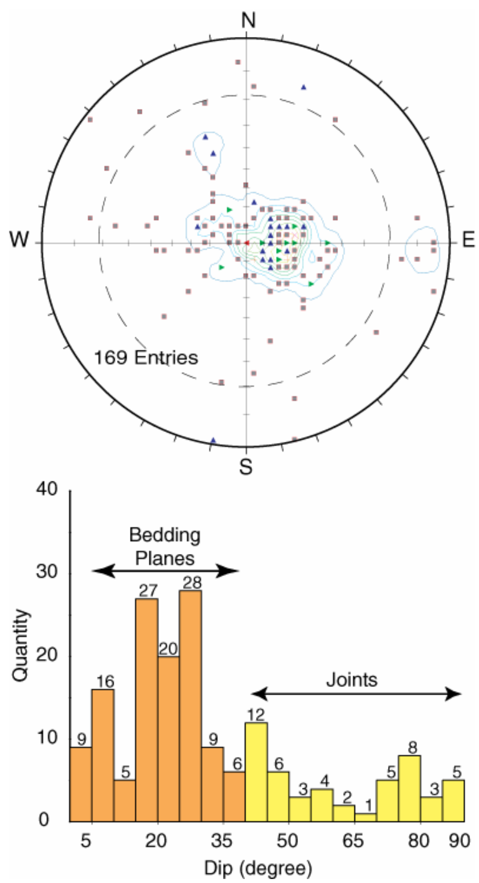

Figure 3: Orientation of all fractures measured in core. Note the majority of fractures are classed as bedding planes.

A laboratory program was carried out in 2001 (Haley & Aldrich, 2001) to determine the strength of the discontinuities. Direct shear tests were carried out on core samples to establish the strength of the bedding planes and Brazilian, uniaxial, and triaxial tests were carried out to establish the shear strength of the intact rock. A summary of the results for open bedding planes is given in Figure 4.

The bedding plane test results were consistent and suggest an average internal friction angle of approximately 30° and a cohesive strength that varied from 68 to 125 kPa with a mean of 96 kPa. The direct shear tests on these open bedding planes showed that there was little difference between the peak and residual strength for most of the samples tested. Hence, if the excavation-induced displacements remained small and blasting procedures were well controlled, the cohesion indicated by the laboratory tests could be preserved near the rock face.

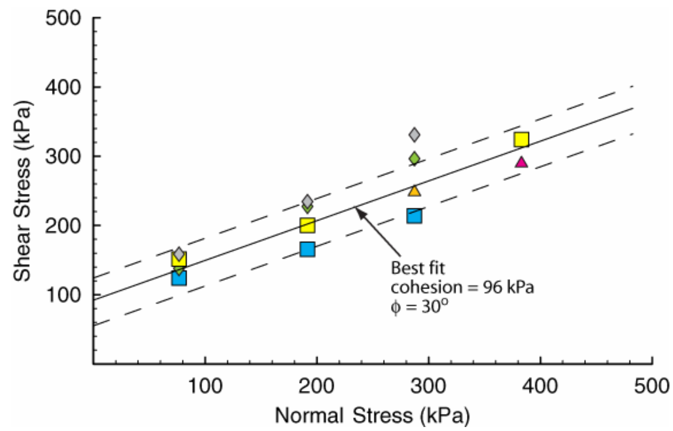

Figure 4: Summary of bedding plane shear strengths measured in direct shear tests on core samples. All samples tested exhibited a discrete open bedding plane

2.3 *Groundwater conditions*

A total of 26 observation wells were installed during multiple phases of subsurface investigations in the vicinity of planned project excavations. The results from these wells indicated that the stabilized groundwater level was at depths between 2 and 4 m; however, groundwater yields and inflow to wells was low. Rising head permeability tests in 15 of these wells and multi-well pumping tests gave hydraulic conductivities between $1x10^{-6}$ and $1x10^{-8}$ m/s, with an average of $3x10^{-7}$ m/s, with the 'permeability' of the rock mass representative of the flow through rock fractures. These hydraulic conductivities indicated that the rock cuts could be adequately drained using conventional horizontal drains drilled from the excavation face. For such situations experience has shown that drain holes drilled on a spacing of 3 to 10 m to a depth of about 1/2 to 1/3 of the slope height provide adequate drainage (Wyllie & Mah, 2004).

2.4 *Summary*

The geology for the site and the results from the site investigation suggested that the bedding planes were the dominant discontinuity that posed the greatest hazard to the proposed rock excavations. In the next section the methodology that was used to evaluate this potential hazard is discussed.

3 EXCAVATION ANALYSES

The most important and essential first step in designing the support for any rock cut is establishing the appropriate geological model for the rock conditions at hand. Geotechnical investigations confirmed that the major structural features that could impact the design of the rock cuts were the bedding planes and subvertical joints. The bedding planes strike approximately North-South while the subvertical joints were random but regionally strike North-South and East-West. As a first step it is reasonable to assume that the failure of the rock cut could occur along: (1) a bedding plane, (2) subvertical joint, (3) a combination of subvertical joints and bedding plane, or (4) blast induced fracturing. An illustration of these various failure modes is illustrated in Figure 5. As can be seen in Figure 5 considerable simplification is required in going from a site geological model to the failure mode model. However, it is important that the essential characteristics of the fractures, particularly regarding the strength and continuity, be incorporated into the failure modes (Figure 5)

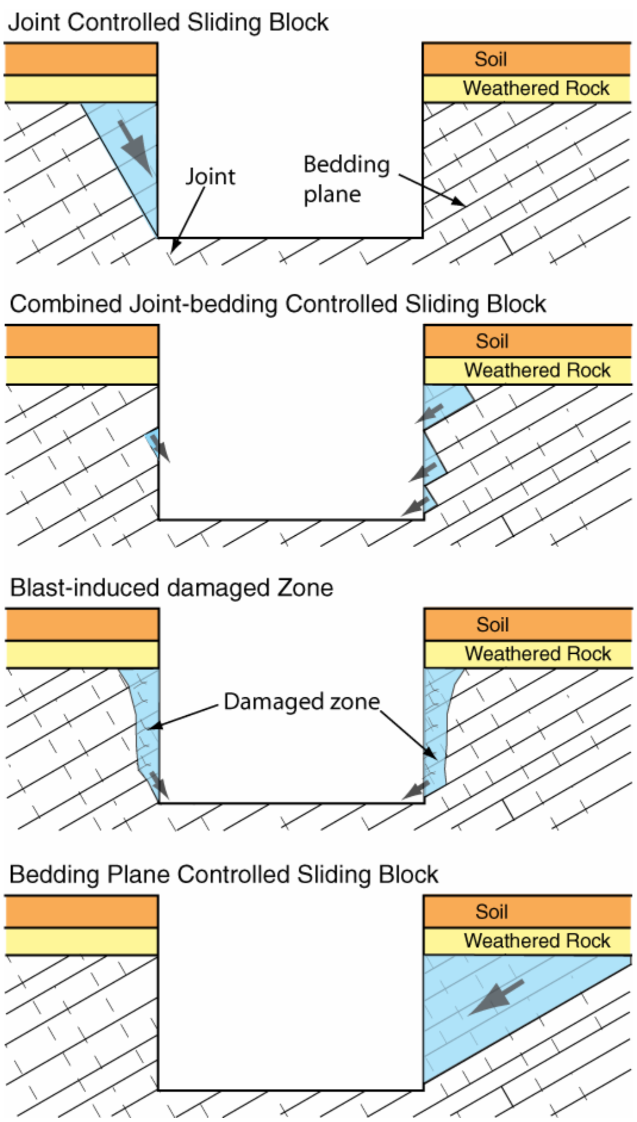

Figure 5: Example of possible failure mechanisms that needed to be considered in designing rock cuts at the Dulles Airport site.

There are two basic methods that can be used to assess the lateral pressures required for design of excavation support for the rock cuts: (1) traditional limit equilibrium method (LEM, see Figure 6), and (2) numerical methods such as finite element or discrete elements. When there is uncertainty in the shear strength, loading parameters, and slope angle, a probabilistic approach can be coupled with the traditional limit equilibrium method. The numerical methods readily available today provide a means of evaluating the construction sequence and support interaction, something which cannot be evaluated using the limit equilibrium method. These numerical methods also provide a link with instrumentation records and the slope performance. However, regardless of the method used to assess the stability of the slope, it is assumed that the geological model is correct.

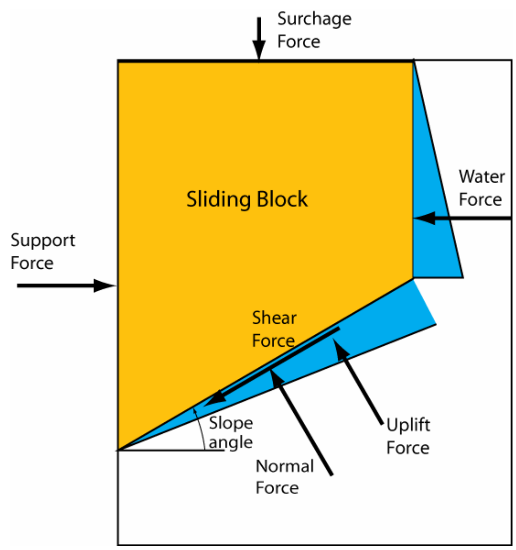

Figure 6: Simple sliding block model used in limit equilibrium analysis.

3.1 *Numerical analysis*

The finite element package Phase2 (available from www.rocscience.com) was used to simulate a deep rock cut in the mudstones and siltstones encountered at Dulles Airport (Figure 7). The model was constructed looking North and included the soil, weathered rock, and the mudstones/siltstones. Bedding planes and joints were included to simulate the bedding planes dipping at 30° and the subvertical joints. The location of the joints and bedding planes was limited to the East and West Wall to reduce the simulation time. For these analyses the slope was assumed to be adequately drained such that uplift pressures could be ignored. The main focus of this study was to:

1. determine the potential shape of the yield zone on the East and West Walls;

2. assess the strength of the joints required to bring the system to equilibrium using the nominal support of 5 tonne 3-metre-long split sets on a 2.1 x 2.1 metre pattern;
3. assess the loads on the bolts at the end of excavation;
4. assess the effect of a 1.5-m thick blast induced damaged zone; and
5. assess the stability of the cut assuming long-term strength parameters for both the bedding planes and subvertical joints. For this purpose, the cohesion of joints and bedding planes was decreased, in stages, from an initial value of 96 kPa to 0.6 kPa. A cohesion value of zero would cause the East wall to fail.

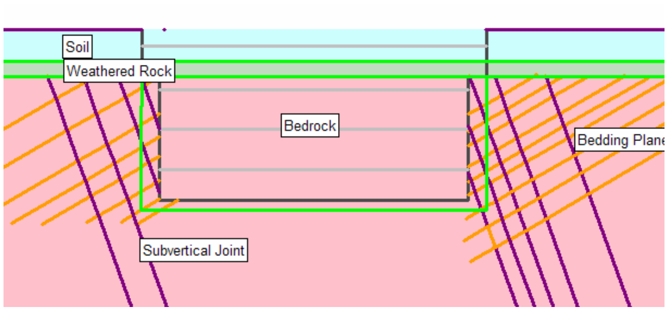

Figure 7: Phase2 model used to simulate the rock cut. The left side of the model is West and the right side

The material properties and strength parameters used in the Phase2 analyses are given below for each of the material types:

Soil:
Modulus=300 MPa, Poisson's ratio=0.4,
Unit weight=0.018 MN/m^3
Friction = 30°, Cohesion = 0.050 MPa

Weathered rock
Modulus=2000 MPa, Poisson's ratio=0.3
Unit weight=0.022 MN/m^3
Friction = 35 °, Cohesion = 0.5 MPa,

Mudstone/Siltstones:
Geological Strength Index= 40,
Modulus=3600 MPa, Poisson's ratio=0.25,
Unit weight=0.026 MN/m^3
Hoek-Brown Failure parameters:
σ_{ci} = 40 MPa , m_b =1.994, s= 0.0013

Blasting-induced damaged Mudstone/Siltstones:
Modulus=2489 MPa, Poisson's ratio=0.3,
Unit weight=0.025 MN/m3
Hoek-Brown Failure parameters:
σ_{ci} = 40 MPa, m_b =0.796, s= 0.0002

Bedding Plane:
Joint stiffness: kn=8000 MPa/m, ks=4000 MPa/m
Friction = 26 °, Cohesion = 0.1 MPa
Tension=0.001 MPa

Subvertical Joint:
Joint stiffness: kn=8000 MPa/m, ks=4000 MPa/m
Friction = 40 °, Cohesion = 0.1 MPa,
Tension = 0.001 MPa

In-situ stress:
A gravity stress state was assumed with the horizontal stress equal to 0.8 of the vertical stress.

The joint stiffness parameters were estimated using the published work of Barton et al (1985) on joint characterization. The frictional strength parameters were based on the laboratory direct shear results contained in the Geotechnical Data Report (Haley & Alrich, 2001) for "open bedding planes". The results were reported as "residual shear strength" suggesting that the peak and residual shear strength was similar, i.e., there is no rapid loss of strength with shear displacement. A least squares fit for the data gave cohesion=96 kPa and friction (φ)=29.8°. Analyses were carried out using φ=30° and φ=26° for the bedding planes. Only the φ=26° are discussed here as these results represent a lower bound for the bedding planes dipping 30° . The results for the bedding planes with φ=30° showed similar trends although the displacements and the corresponding loads on the rock bolts were slightly less.

It was assumed that, because the shear strength of the bedding planes included some cohesion, movements induced by the excavation and blasting could reduce this cohesion. To simulate this effect and the potential for cohesion-loss with time as a result of softening, the cohesion strength component was reduced to a very low value (0.7 kPa) in a series of stages for both the bedding planes and the subvertical joints.

In the analyses, split set anchors were staged with each level of excavation and installed as the excavation proceeded. The anchor pattern in the model was based on a 2.1 m x 2.1 m pattern and the assigned pullout capacity of the Ingersoll-Rand SS-46 split sets was set to 49 kN.

A total of 17 stages were used in the analyses. Stage 1 was used for initial equilibrium and Stages 2 to 8 were used for excavation and installation of the support (Figure 8). Stage 9 was the application of a surcharge of 12 kN/m to the top of the slope and Stage 10 was the application of the blast-induced damage. This was uniformly applied to

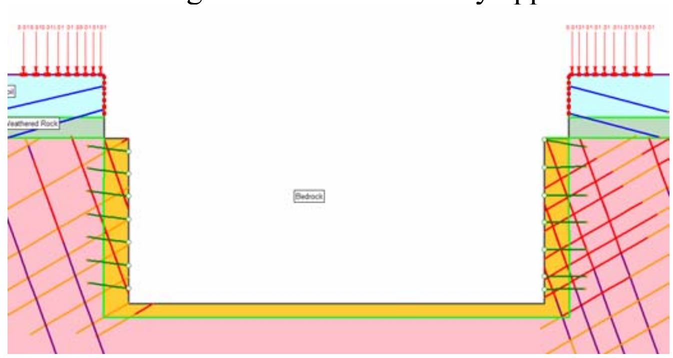

Figure 8: Phase2 model at the end of excavation with the anchors installed.

the whole slope to assess the impact of blast damage on slope displacements and for comparison to a slope where blast-induced damage was eliminated through blast-control measures. Stages 11 through 17 were used to reduce the cohesion from 96 kPa to 0.6 kPa. In all the analyses, the dip of the bedding planes was assumed to be at 30 ° and uniform.

Figure 9 shows the results from the analyses at the end of excavation and after the cohesion had been reduced to 0.6 kPa. The individual anchor loads of 5.3 to 20.9 kN do not exceed the anchor capacity of 49 kN. Minor yielding has occurred along some of the joints and bedding planes but this zone is restricted to a wedge inclined approximately 60° (see dashed line in Figure 9). The potential for slip is enhanced in this wedge shaped region because of the maximum unloading that occurs in this zone. Slip along the bedding planes and subvertical joints could lead to a stepped-shape failure surface, generally following the 30° bedding and the subvertical joint, even though there is no single plane for sliding. Excavation support must be adequate to minimize the disturbance to the rock mass in this zone and to minimize the size of a potential failure zone if the subvertical joints control the mode of failure.

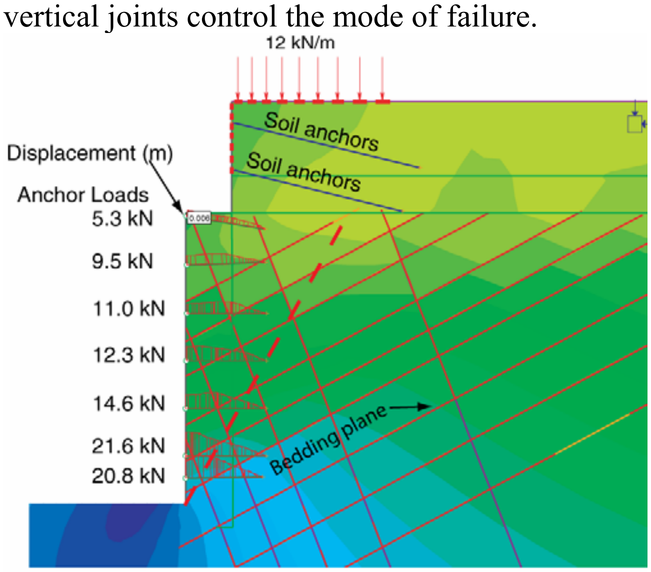

Figure 9: East Wall – Stage 17 showing anchor loads and yielding in joints. Note that even though the cohesive strength is only 0.6 kPa there is still considerable reserve capacity in the bolts (split sets usually have a pull-out capacity of approximately 49 to 68 kN). The bedding is dipping into the excavation at 30 °.

Figure 10 shows the reduction in cohesion at each stage in the numerical analysis versus the anchor load normalized to the anchor capacity. At the end of construction, and assuming limited blast induced damage occurs, no instability is observed along any of the joints or bedding planes. The Triassic red beds encountered at the Dulles Airport site are known to deteriorate when exposed to the natural elements. Therefore, the rock cuts were covered with shotcrete to minimize the potential for deterioration. Because of this weathering sensitivity and the potential for a loss in cohesion during and after excavation the cohesion was gradually reduced to assess the impact on anchor loads. As the cohesion is reduced there is only minor increase in the anchor load until about 8 kPa (Figure 10). However, even with 0.6 kPa cohesion the anchor loads are still well below the anchor capacity. Reducing the cohesion to zero results in complete failure along the deepest daylighting bedding plane (see Figure 9).

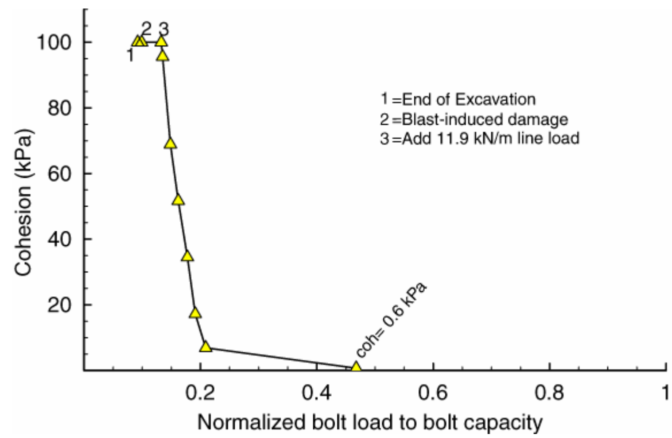

Figure 10: The reduction in cohesion along the bedding planes and the corresponding load increase in the anchor bolts from the numerical analyses.

4 CONSTRUCTION PERFORMANCE

Today there have been several thousand lineal metres of rock cuts at the Dulles Airport site, varying in height from 5 to 25 m. All of the cuts in rock have been vertical and excavated using either blasting, saw-cutting, or roadheader techniques (Figure 11). In excavations adjacent to buildings and equipment blasting is very carefully controlled.

Figure 11: Typical rock cuts at Dulles Airport site (Tier 3 Excavation, photo from McQuinn et al 2006)

4.1 *Construction Sequence*

The excavation sequence for rock cuts must be compatible with the anticipated rock response and typical construction equipment. The sequence involves excavating the bench which is typically 2 to 3 m high, installing the support, and installing the drainholes. When the rock face is supported with shotcrete, the support requires several additional steps depending on the type of shotcrete used, e.g., plain or fibre- or mesh-reinforced. It is important in the excavation sequence to ensure that the support is installed before the next bench is excavated. As construction equipment increases in size and capacity there is a tendency to excavate double or triple benches. In critical excavations such as the Dulles Airport site, controlling the bench height controls the displacements, which in turn controls the stability. In the numerical analysis in the previous section, it was shown that stability was readily maintained if the cohesion was not destroyed. One way to minimize the loss of cohesion is to control the displacements, which are related to the bench height. For the excavation of the rock cuts at the Dulles Airport site, bench heights are restricted to 2 to 3 m.

In one excavation, a double bench height was taken and this extra bench height resulted in localized failure along a bedding plane (Figure 12). Inspection of the failure suggested that the instability was bounded by the bedding planes and the subvertical joint planes, similar to the mechanism shown in Figure 5. Periodically, smaller (<4m^3) wedge falls have formed, but these have typically formed near horizontal and vertical corner excavations where confining stresses have been reduced.

Figure 12: Localised failure along a bedding plane when a double-height bench was excavated.

4.2 *Drainage*

Construction drainage is considered standard practice for rock excavations below the groundwater table. Drainage is provided via drainholes that are drilled as excavation advances. Drainholes not only reduce the uplift pressures near the face, but also change the direction of the seepage force, which can also improve stability. The effect of drainholes on the water table can be observed in the construction of the excavations for the Taxiway F APM tunnel crossing. At Taxiway F, 6 piezometers were installed about 9.1 m behind the excavation face. These piezometers were sealed through the soil overburden and into weathered rock to a depth of about 4 m, and were screened to about 1 meter below the level of the bottom of excavation. Excavation operations lasted from late April to the first part of October 2002. During the excavation 4 rows of drain holes were installed. These rows were spaced 1.5 m vertically and the drains in each row spaced 6.5 m horizontally. The first row was 15.25 m long, the second row 12 m, third row 9 m and the bottom row 6 m long.

The response of the piezometers to excavation and drainhole drilling is clearly seen in Figure 13. The ground water levels in piezometers on the East side dropped approximately 5 m by the completion of excavation, while those in the piezometers on the West side dropped approximately 3 m.

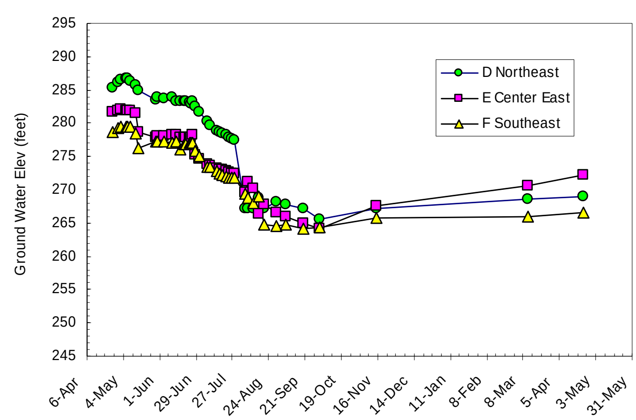

Figure 13: Ground water levels recorded by the piezometers on the East side of Taxiway F excavation.

4.3 *Excavation monitoring*

In keeping with the observational method, rock cuts must be monitored. The instrumentation at the Dulles Airport site includes automated in-place inclinometers, automated EDM surveys, and extensometers. Instruments are routinely monitored during excavations and results conveyed to the contractor and project office. Threshold displacement limits were established for each excavation site and warning levels assigned. All instruments have shown that wall deformations are well within the expected limits.

4.4 *Excavation mapping and testing*

During the course of the excavations, detailed mapping was carried out to confirm the design assumptions. The major concern was the dip of the bedding and the potential for large-scale bedding planes infilled with clay, particularly in the upper part of the rock mass below the weathered zone. Figure 14 gives the results from detailed field mapping of several rock cuts and confirms the results from the original site investigation (see Figure 3). In addition 100-mm diameter samples of the bedding planes were collected and tested in direct shear (Schnabel Engineer, 2006). These results were also within the previous design assumptions.

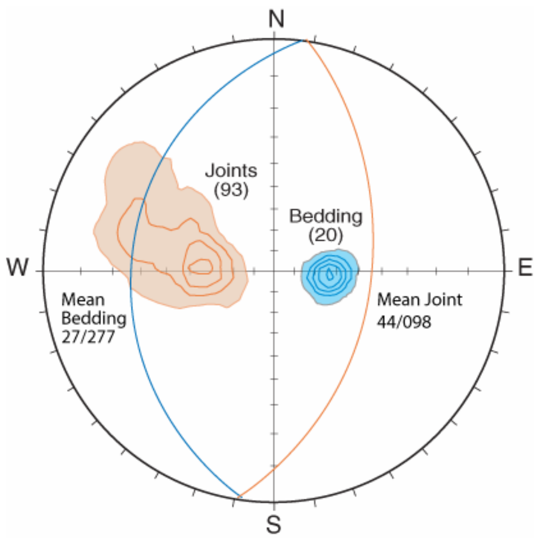

Figure 14: Lower hemisphere stereonet of the field mapping of the rock cut giving the large scale dip of the bedding and the joints.

5 CONCLUSIONS

Deep rock cuts for below-ground for the APM tunnel stations at Dulles Airport presented several challenges. The approximately 30° dip of the bedding planes into the 25 m-deep excavation presented a significant potential hazard. A worst-case scenario would require supporting a sliding block that daylighted at the bottom of the excavation. Such a support system if adopted for the whole site would have had significant cost and scheduling implications. Through a detailed site investigation and laboratory testing program combined with extensive analyses, it was concluded that the probability of large scale bedding-plane-controlled instabilities was low. As a result, the permanent station walls were designed to contain the wedge of rock that was considered to be the most likely mode of failure. This reduced lateral support pressure of 12 kPa was less than 1/10th the support system that would be required for a large bedding-controlled block. Extensive field mapping and a comprehensive monitoring system have confirmed the design assumptions. Future excavations will continue to implement the observational design method.

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