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Support of Rock Cuts at Washington-Dulles International Airport

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SUPPORT OF ROCK CUTS AT WASHINGTON-DULLES INTERNATIONAL AIRPORT

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ABSTRACT

Expansions at the Washington-Dulles International Airport since 1999 have required extensive vertical, open-cut rock excavations in Triassic age siltstone bedrock. These excavations have extended to depths of up to approximately 65 ft (20 m) adjacent to existing infrastructure for construction of new below-ground stations for the new Automated People Mover (APM) light rail system. The selection of design support pressures for the rock excavations was an important decision, balancing the projects' risks and construction costs. At the center of this issue was the development of a geotechnical model of the rock mass and its primary failure mechanism. Thus, a comprehensive subsurface characterization was required. The rock mass characterization included observation and mapping of excavation faces, detailed logging of rock cores, use of optical and acoustic televiewer, testing of discontinuity samples for shear strength evaluation, groundwater monitoring, and inclinometer monitoring of supported faces.

The televiewer data, combined with site observations, allowed for a more complete understanding of the engineering characteristics of the bedding plane and joint discontinuities within the siltstone rock mass. Based on the pattern of the predominant discontinuities, it was concluded that bedding planes dipping into the excavation at approximately 30 degrees intersecting near-vertical joints would present the greatest risk for rock cut failures. Extensive laboratory testing and field inspections at a variety of exposed cuts with varying bedding plane and joint orientations suggested that the potential for a large slide along a bedding plane was relatively low. This conclusion was based on observations of discontinuous clay seams of limited number, the first- and second-order roughness of joint and bedding plane surfaces, and the limited persistence of joint and bedding plane discontinuities. Previous design lateral pressures for permanent station walls had been based on an assumed potential failure model of a large, excavation-scale block failure. However, using the recent characterization data, the rock mass failure mechanism of a local joint- and bedding-controlled sliding block mechanism was considered more appropriate. The resulting design lateral pressure necessary to support a rock face using this mechanism and the shear strength of discontinuities and intact rock was significantly lower than the initial design values. Construction-phase observations and monitoring, which included detailed field mapping, automated instrumentation monitoring, and groundwater monitoring, have verified the rock characterization and design assumptions.

The reduction in design pressures for the permanent below-grade walls for the APM station structures resulted in major cost savings for the projects now in design and construction. Based on the scale of future expansion plans at Dulles, the projected total cost savings resulting from the reduced design lateral rock pressures will be considerable.

INTRODUCTION

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 65 feet (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 recommended values of rock support pressures provided by engineers involved with the project varied significantly. Due to the scale of the project, the construction cost implications associated with support measures using the rock pressure estimations in design were significant.

The basis for the design rock pressure value was developed through the observational approach. 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 investigation, the design sequence, the recommended support pressures, and the results from constructing several cuts.

ROCK MASS CHARACTERIZATION

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. Most rock masses lie between these two end members, and the support requirements can vary significantly.

The rock mass at Dulles Airport is a siltstone and mudstone that can range in appearance from thinly bedded to blocky when excavated. The bedding planes are the obvious and controlling discontinuity. Thus, characterization of the bedding planes was a critical issue to the rock mass characterization. In general, sedimentary rocks are characterized by transversely isotropic layers. Although the calculations to evaluate discontinuities are straightforward, the challenge lies in assessing the strength and continuity (persistence) of the bedding planes and rock joints. Although a traditional site investigation had been performed at the Washington-Dulles International Airport (Dulles Airport) expansion project (Figure 1), the investigation was not adequate to evaluate the orientation, strength and persistence of the discontinuities to understand their influence on excavated rock mass stability.

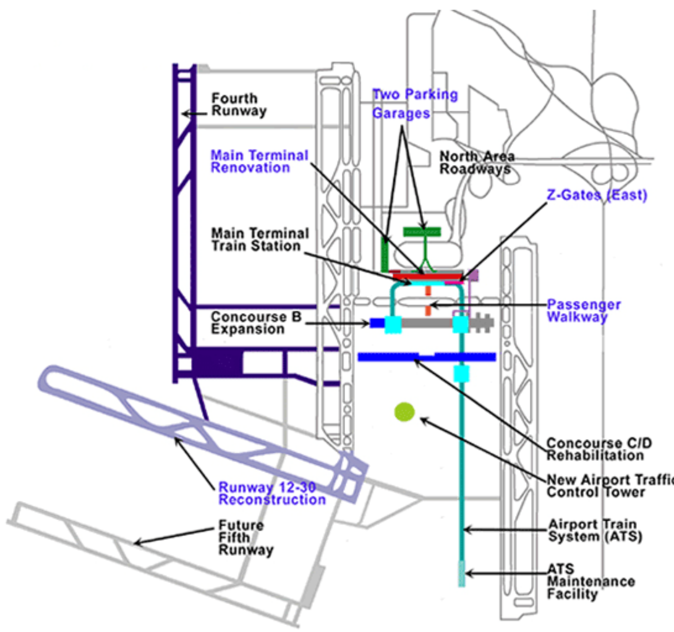


Figure 1: Major projects map for Washington-Dulles International Airport, *www.mwaa.com/dulles/d2_dulles_development_2/d2_home*

Rock Mass Investigation

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.

Geological Setting

The project site lies within the Culpeper Basin, a Triassic Sedimentary basin that extends from South Central Virginia to western Maryland. This sedimentary series was caused by the continental rifting that later resulted in the opening of the Atlantic Ocean (Southworth et al, 2006). The predominant rocks of the Culpeper Basin are mudstones, siltstones, shales, and sandstones. At the Dulles Airport site, the bedding is generally about one to several inches thick, though thicker beds appearing as blocks up to several feet in thickness sometimes appear. 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). The regional deformation history experienced by these rocks pointed toward non-persistent fractures (Price and Cosgrove, 1990). While the region has been uplifted and intruded by igneous rocks, the sedimentary sequence is relatively undeformed. Previous 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. The persistence of the bedding planes and sub-vertical fractures was a critical question to be considered in this study as the implications for lateral earth pressures were significant.

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, and seismic refraction surveys. The 2006 study included rock cores with optical and acoustic televiewer testing and laboratory testing of both intact cores and discontinuity samples. The key characterization data obtained in this study concerned discontinuity orientation and persistence, the shear strength of discontinuities, and evaluation of groundwater conditions. This paper focuses on those aspects of the characterization.

Discontinuity Characterization.

To better evaluate the orientation, nature, and persistence of discontinuities, this study used optic and acoustic televiewer surveys in conjunction with rockwall mapping and detailed review of rock cores logs. The rock cores obtained generally had high (greater than 98%) recovery and RQD of 70 or greater. The televiewer was selected for use as it is a valuable tool for observing undisturbed discontinuities and in-situ estimation of apertures. It also permitted an evaluation of the persistence of some of the larger discontinuities observed on the rock face by projection. Based on observations and correlation with televiewer data, most of the discontinuities identified in the borings were non-continuous or non-flow features, and bedding plane features that appeared significant at the open cuts were not observed to extend some distance within and/or from the excavation face. The separation of bedding and joint features indicated that only 13 percent of the features had apertures of about ½-inch (10mm) or greater. Only five features of the 250+ ft (76+ m) of rock cored had apertures with up to about 1 inch (20 mm) of separation. A concern was that clay infilling was prevalent in bedding plane discontinuities. However, only a fraction of the discontinuities had more than minor clay infilling observed in cores with the televiewer, and where observed, thicknesses were generally about 1.5 mm (1/16-inch) or less. Calcite was observed in some discontinuities.

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 was challenging to obtain good core recovery and undisturbed samples. Both intact and discontinuity samples were tested in a rock mechanics laboratory.

The investigations concluded that the discontinuities along bedding planes posed the biggest threat to the stability of the planned excavations, as they dipped mainly between 15 to 35° towards the west. Figure 2 shows the distribution of discontinuity feature by orientation for features that were continuous and large enough to permit possible water flow.

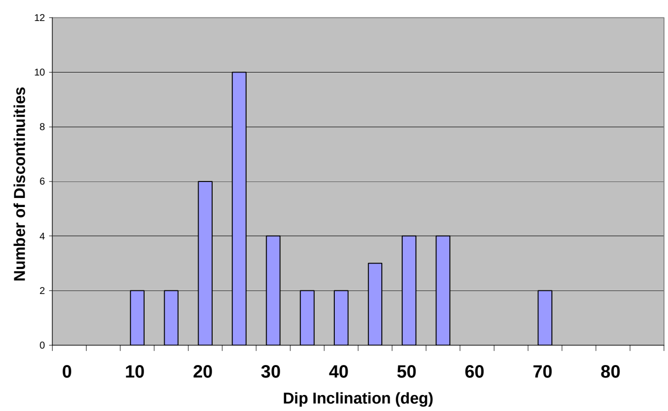


Figure 2: Distribution of Discontinuity by Inclination

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 developed based on the televiewer work (Colog, 2006).

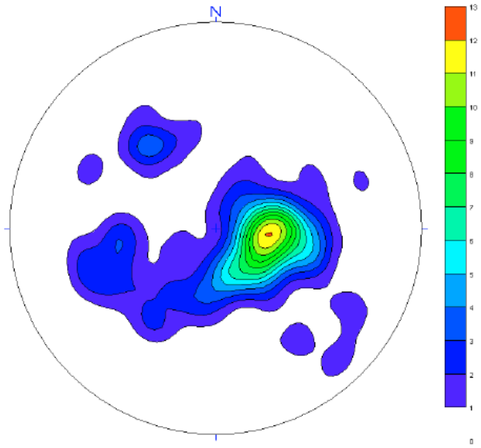


Figure 3: Orientation of poles to all discontinuities measured by televiewer projected on a lower hemisphere stereonet. Note the majority of fractures are classed as bedding planes.

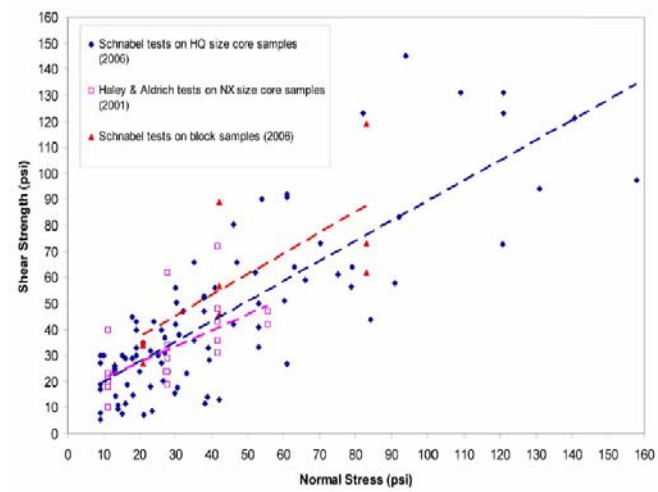
Based on these investigations it was decided that an average bedding plane dip of 30° would be used for the design of the open cuts. It is clear from both Figures 2 and Figure 3 that the orientation of the joint sets is less well-defined. However, the steeply-dipping features can be difficult to observe with both vertical and subvertical boreholes.

<u>Discontinuity Shear Strength.</u>

A laboratory testing program was carried out in 2001 (Haley & Aldrich, 2001) to determine the strength of the discontinuities. Direct shear tests were carried out on 11 core samples to establish the strength of the bedding planes were carried out on NQ (1-3/8 in or 35mm dia.) cores to establish the shear strength of the discontinuities. To supplement this data, the 2006 study directed a testing program including 35 HQ (2-1/2 or 63.5 mm dia.) core and three larger 4-inch (100mm) diameter discontinuity block samples.. The discontinuities were tested by the direct shear method (ASTM 5607). Evaluation of the results considered whether the samples represented bedding or joint discontinuities, the size of the sample tested, and whether clay infilling was observed in the discontinuity.

The study found that the bedding discontinuities had higher mean friction angles (37°) in comparison to the joints (27°) with similar mean cohesion intercept values (16.4 for joints v. 14.6 psi for bedding, 113 kPa v. 100 kPa). The samples with clay infilling indicated mean friction angles of 38° deg with cohesion intercept values of 6.8 psi (47 kPa). Not surprisingly, the larger samples indicated slightly higher shear strength values with mean friction values of 39°, 34°, and 30° for the block, HQ, and NQ core samples tested. The undulations in bedding planes that were observed in rock cuts as first-order discontinuities could not be represented in the testing scale. However, the undulations of smaller-order discontinuities developed greater shear strength with increased shearing surface.

A summary of the results for the direct shear results is given in Figure 4. The test results were consistent and suggest an average internal friction angle of approximately 30° and a cohesion intercept that varied from 9.8 to 18.1 psi (68 to 125 kPa) with a mean of 13.9 psi (96 kPa). Although the cohesive strength is not present in open fractures, some cohesion contribution can be considered due to the non-persistence of the discontinuities. The direct shear tests on these open bedding planes showed that there was little difference between the peak and residual (post-peak) 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.



NOTE: Trend lines were produced using linear best fit function in Microsoft Excel 2003

Figure 4: Summary of bedding plane and joint sample shear strengths measured in direct shear tests on HQ and NQ core and block samples. All samples tested exhibited a discrete open bedding plane

<u>Groundwater Conditions.</u>

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, where present, was at heights between 10 and 15 feet (3 to 4.5 m) above the excavation face. 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. Since the depth of the excavations extended up to about 65 feet (20 m), the head was significant. However, groundwater was only rarely observed and where observed, was seen as seeps and was easily handled with sumps. This evaluation was critical, as the influence of water in the rock mass stability is great.

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. Without excavation support, the primary failure mechanism for the west-facing cuts was expected to be block sliding. On the east-facing cuts, the primary failure mechanism expected was soil-type slumping to use terminology consistent with Goodman and Kieffer (2000). Excavation support with tock bolts and split sets was planned. In the next section the methodology that was used to evaluate this potential hazard is discussed.

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 less predictable 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).

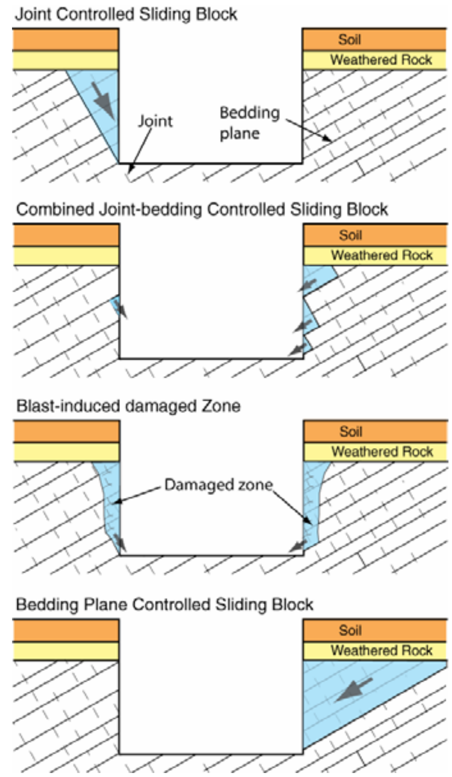


Figure 5: Example of possible failure mechanisms that needed to be considered in designing rock cuts at the Dulles Airport site.

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.

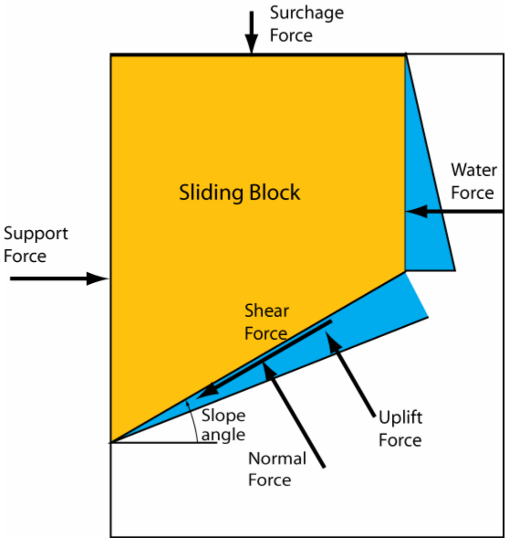


Figure 6: Simple sliding block model used in limit equilibrium analysis.

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 7 x7 ft (2.1 x 2.1 m) pattern;
3. assess the loads on the bolts at the end of excavation;
4. assess the effect of a 5 ft (1.5m) 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.

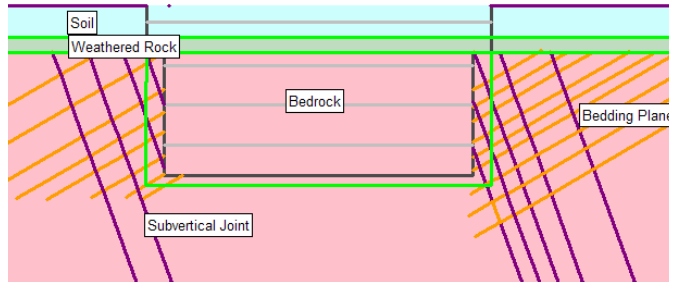


Figure 7: Phase2 model used to simulate the rock cut. The left side of the model is West and the right side

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

Soil:
Modulus=43.5 ksi (300 MPa), Poisson's ratio=0.4,
Unit weight= 125 pcf (0.018 MN/m^3)
Friction = 30°, Cohesion = 7 psi (0.050 MPa)

Weathered rock
Modulus= 290 ksi (2000 MPa), Poisson's ratio=0.3
Unit weight= 151 pcf (0.022 MN/m^3)
Friction = 35 °, Cohesion = 0.5 MPa,

Mudstone/Siltstones:
Geological Strength Index= 40,
Modulus=522 ksi (3600 MPa), Poisson's ratio=0.25,
Unit weight=180 pcf (0.026 MN/m^3)
Hoek-Brown Failure parameters:
σci = 5.8 ksi (40 MPa) , mb =1.994, s= 0.0013

Blasting-induced damaged Mudstone/Siltstones:
Modulus=360 ksi (2489 MPa), Poisson's ratio=0.3,
Unit weight=172 pcf (0.025 MN/m^3)
Hoek-Brown Failure parameters:
σci = 5.8 ksi (40 MPa), mb =0.796, s= 0.0002

Bedding Plane:
Joint stiffness: kn=361 ksi/ft (8000 MPa/m), ks=180 ksi/ft (4000 MPa/m)
Friction = 26 °, Cohesion = 14.5 psi (0.1 MPa)
Tension=0.1 psi (0.001 MPa)

Subvertical Joint:
Joint stiffness: kn= 361ksi/ft (8000 MPa/m), ks=180 ksi/ft (4000 MPa/m)
Friction = 40 °, Cohesion = 14.5 psi (0.1 Mpa),
Tension =0.1 psi (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. The laboratory results indicated that the peak and residual (post-peak) 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=14 psi (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 (0.1 psi or 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 6.75 x 6.75 ft (2.1 x 2.1 m) pattern and the assigned pullout capacity of the Ingersoll-Rand SS-46 split sets was set to 11 kips (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 0.8 kip/ft (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

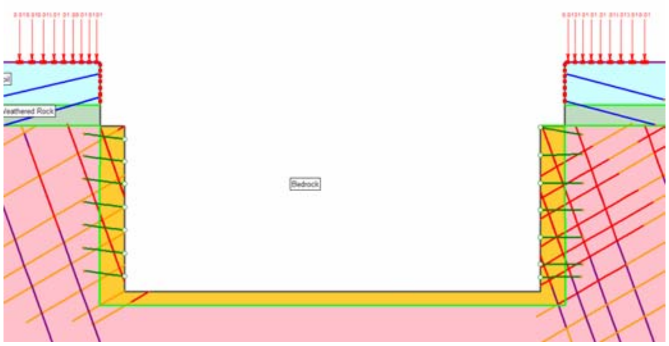


Figure 8: Phase2 model at the end of excavation with the anchors installed.

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 14 to 0. 1 psi (96 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 11 kips (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.

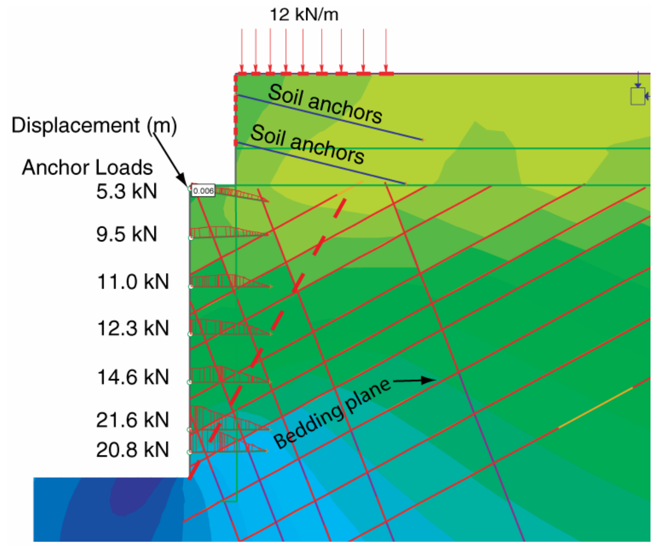


Figure 9: East Wall – Stage 17 showing anchor loads and yielding in joints. Note that even though the cohesive strength is only 0.6 kPa there is still considerable reserve capacity in the bolts (split sets usually have a pull-out capacity of approximately 11 to 15 kips/ 49 to 68 kN). The bedding is dipping 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 1.2 psi (8 kPa, Figure 10). However, even with 0.1 psi (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).

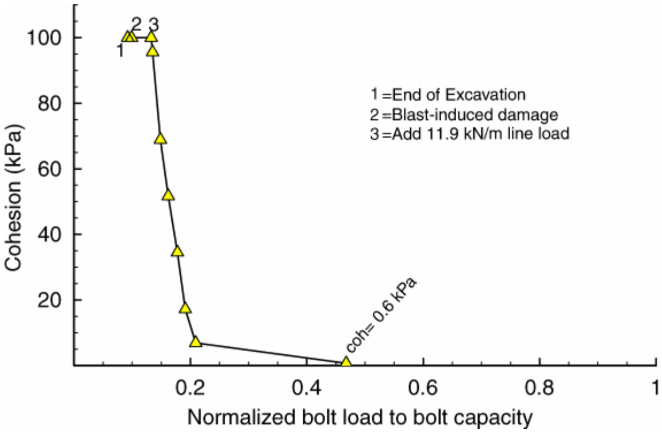


Figure 10: The reduction in cohesion along the bedding planes and the corresponding load increase in the anchor bolts from the numerical analyses.

CONSTRUCTION PERFORMANCE

Today there have been several thousand lineal feet of rock cuts at the Dulles Airport site, varying in height from 16 to 65 ft (5 to 20 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)

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 6.5 to 10 ft (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 6.5 to 10 ft (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 sub-vertical joint planes, similar to the mechanism shown in Figure 5. Periodically, smaller (<3yd^3 or 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: Localized failure along a bedding plane when a double-height bench was excavated.

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 30 ft (9.1 m) behind the excavation face. These piezometers were sealed through the soil overburden and into weathered rock to a depth of about 13 ft (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 5 ft (1.5 m) vertically and the drains in each row spaced 20 ft (6.5 m) horizontally. The first row was 50 ft (15.25 m) long, the second row 40 ft (12 m), third row 30 ft (9 m) and the bottom row 20 ft (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 16 ft (5 m) by the completion of excavation, while those in the piezometers on the West side dropped approximately 10 ft (3 m.)

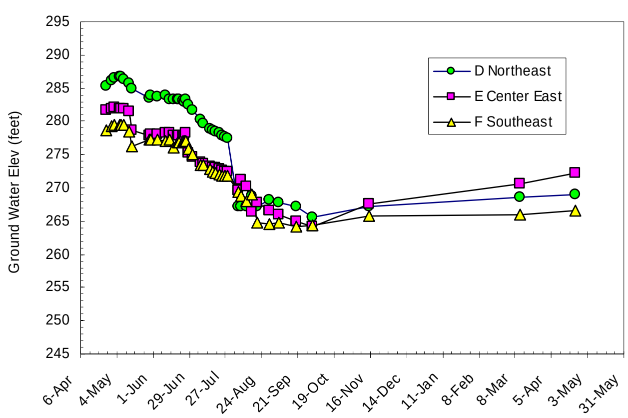


Figure 13: Ground water levels recorded by the piezometers on the East side of Taxiway F excavation.

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.

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 block diameter samples of the bedding planes were collected and tested in direct shear as discussed above. These results were also within the previous design assumptions.

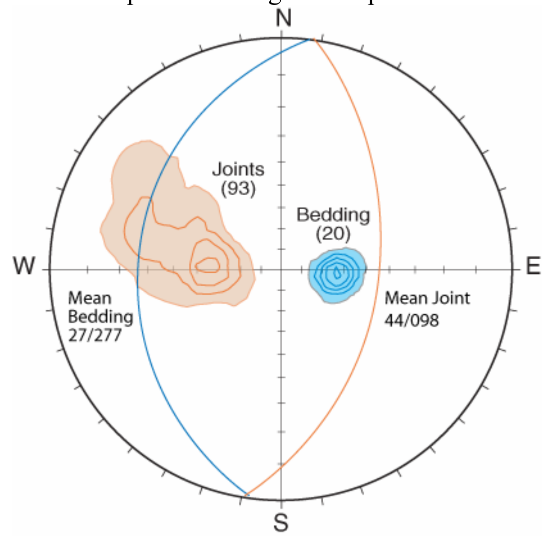


Figure 14: Lower hemisphere stereonet of the field mapping of the rock cut giving the large scale dip of the bedding and the joints.

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 65 ft (20 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 1.7 psi (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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