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GEOTECHNICAL AND DESIGN REVIEW MEMO ON LBNF 100% PRELIMINARY DESIGN BY ARUP

At the end of September 2015, Fermilab engaged Tonon USA, LLC to provide service as a consulting reviewer for the LBNF Director's CD-3A review. The scope of work included serving as a reviewer of the underground conventional facilities aspects of LBNF during a three-day review, and preparing a report as follows:

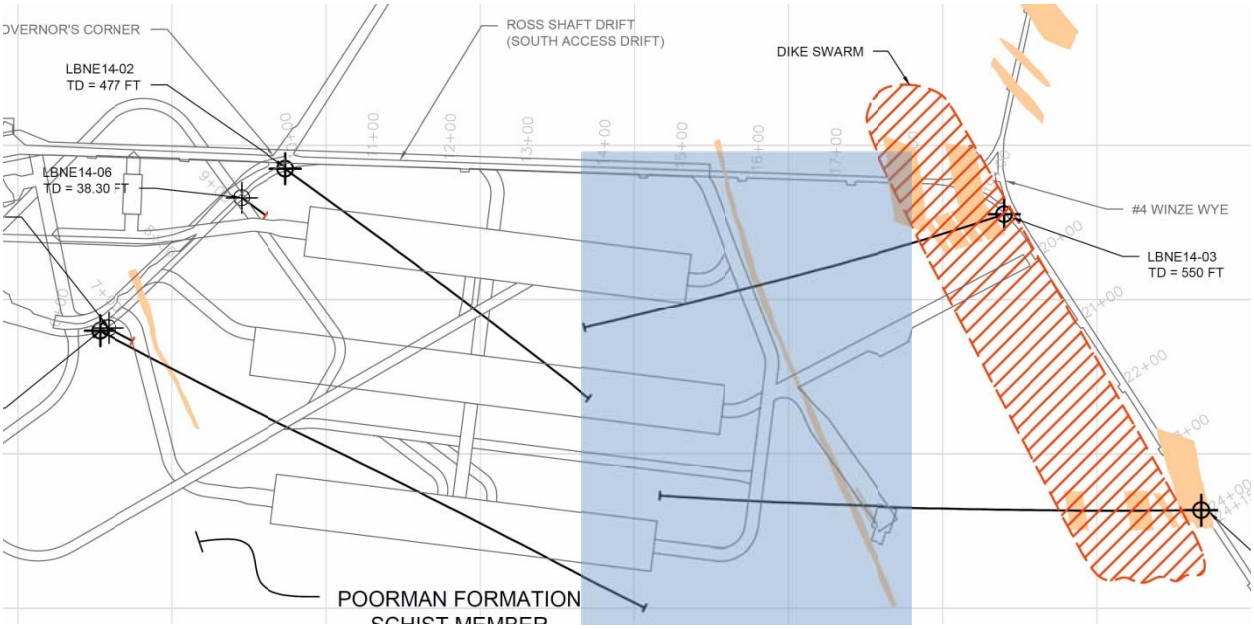
a) Review "read ahead" material as provided by the project prior to the review in order to be prepared for a dense set of information that will be presented by the project at the on-site review event.
b) Attend telephone meetings as needed in preparation for the review.
c) Attend the review for 3.5 days October 26-29 at Sanford Underground Research Facility in Lead, South Dakota. A tour of the underground facility will be conducted on October 26, with the review conducted from the morning of October 27 to mid-day October 29.
d) Contribute underground conventional facilities related findings, comments, and recommendations to the review report. A draft report is due at the conclusion of the review, with the final report due one week later.

Despite the reduced amount of hours (50), I reviewed: the Arup 100% Preliminary Design Report and Drawings (Aug2015), Arup Geotechnical Interpretive Report for 100% Preliminary Design (Aug2015), Arup LBNE Phase 2 Geotechnical Site Investigation Data Report (Sep2014), and Golder In Situ Stress Measurement report for DUSEL (Jan2010). Based on my review, I shared my notes with my fellow reviewer Kevin Hachmeister of Golder Associates, and then with the design team. On Wednesday, October 28, a meeting was held with the design team (including ARUP's internal reviewer), partially present in person at the Sanford Underground Research Facility in Lead, South Dakota, partially participating via conference call.

The following is an edited and revised version of my notes. Cross-references to the abovementioned documents are provided in form of Section cross-references (e.g., 2.3.4) or page numbers of the pdf document (this may be different from the page number printed on the document; using the pdf page number was necessary in order to uniquely cross-reference appendices).

Comments on Arup 100% Preliminary GIR for LBNF

1) 2.3.4 Faults and shears are present in "a particularly deformed rock mass section between approximately Station 13+75 and 17+72 along the Ross Shaft drift".[…] "Several intensely foliated / shear zones were also mapped in the deformed section noted above. These features were often mica-rich and typically parallel to the foliation, making it difficult to identify offsets. Although widely distributed, these features do have significant persistence and represent weak zones within the rock mass". Faults, shears and intensely foliated/shear zones were not taken into account in the design, and there is no design prevision to deal with them during construction. This is not acceptable, and must be corrected because the area affected by faults and shears (shown below in blue) includes a significant portion of the Eastern portions of the caverns and associated drifts.



2) 2.4.1 Spacing was only evaluated for the joints listed in Table 2.1; foliation was NOT included. Foliation is the single most distinctive geologic feature of the rock at the proposed cavern site and must be accounted for in the structural mapping, otherwise, as it stands now, it will not be included in the rock mass classifications and their use in the empirical design. On the other hand, in the rock mass analyses, foliation must be characterized from the strength and deformability viewpoints and explicitly modelled to take into account its directionality. The following figure, taken during the site visit along the Ross Shaft Drift (South Access Drift) on Oct. 26, clearly shows the spacing of the foliation.

3) 2.4.1 The joint spacing data collection suffered from major bias caused by the direction of scanlines, which were mainly oriented E-W. This bias must be corrected to properly account for joint spacing.
4) 2.4.2 Persistence: are these values limited by censoring caused by the dimension of the drift? "Actual measurement of the individual plane area is difficult due to the nature of observation from within a narrow drift". However, in the wedge analysis, an arbitrary persistence of 6 m is used. How was this value obtained when drifts are much smaller than 6m?
5) 2.6 Groundwater. Hydraulic conductivity is low, BUT water pressures are high: 58 to 850 m head. This water pressure builds up quickly: water pressure reading in borehole "LBNE14-2 was discontinued due to safety concerns associated with potential failure of the well head". There is no design prevision to deal with such pressures. Dykes are present in domain 4 and especially 5: are they water barriers?
6) 3.1.2 Only 12 UCS tests were performed in the schist (as opposed to much more test data available on isotropic rhyolite):
    a. It is not specified which of the 3 different schists was tested; Page 46 of GDR makes this distinction, though.
    b. No characterization of the anisotropy of the schist(s) at this stage, then Figs. 24-27.
7) Use of rock mass classifications. The rock mass under consideration is characterized by:
    a. (Section 2.4.1) Fracture clusters spaced at about 10 m; within each cluster, fractures have about 1 m spacing.
    b. Strongly foliated rock, where the foliation is composed of mica and/or graphite minerals, both of which are very low shear strength (graphite is used as a lubricant!). Core inspection during the site visit and the review of the core box pictures in the GDR show that foliation is either planar (most of core length) or contorted (smaller portion of rock core)

    In this situation, the blind use of RQD and rock mass classifications in this project, and their use to justify the presence of an excellent rock mass is questionable at best and leads to erroneous design conclusions because rock mass classifications (as applied in this project) have been developed to characterize jointed rock masses where the rock mass behavior (at the scale of the excavation) is controlled by the fractures, and where 3-4 fracture sets exist in various orientations, so that the rock mass properties (at the scale of the excavation) are isotropic. Recall that cavern construction will occur in stages, where small drifts will be constructed and then enlarged, so the size of excavation varies from a few meters to 20 x 30 m. Here are the thoughts of the fathers of RMR and Q-system on this matter. "If discontinuities are widely spaced or if the intact rock is weak and altered, the properties of the intact rock may strongly influence the gross behavior of the rock mass. Furthermore, a sample of a rock material sometimes represents a small-scale model of the rock mass, since they both have gone through the same geological cycle." (Bieniawski, 1993). "It is unfortunately easy to forget the consequences of anisotropic rock material and rock mass

properties when contemplating the use of rock mass classification methods to derive ballpark input data for numerical models or for empirical design. In the laboratory sample, foliation and schistosity and sedimentation layering may each give anisotropic E-moduli, anisotropic Vp, and quite variable ratios of sigma_c/I50 or compressive to point load tensile strength" (Barton 2002). Although modifications to RMR and Q are available to at least partially capture some of the anisotropy known to affect rock material and rock masses, these modifications have not been used in this project. As a result, the key geologic features of the rock have not been considered in the analysis and design.

8) 3.2.2 Rock Mass Rating (RMR) "The reported RMR does not consider the orientation of discontinuities". Why? This is an integral part of the RMR system: are the orientations variable in the rock mass? Then one should consider one orientation at a time, and determine the variability of RMR caused by fracture orientation. In addition, if RMR is used to judge the "quality" of the rock mass, and in the empirical design, then foliation must be included in the "discontinuities". The orientation of foliation will be a major player in the stability of the rock mass, including during construction, where one face will advance against dip, and one will advance with dip => very different rock mass response to the same excavation process.
9) 3.4.2 D=0 is assumed. Very optimistic for schist, especially for this micaceous and graphitic schist that will likely split along the foliation planes during blasting. During the core inspection, mechanical breaks due to core handling were noticed; in addition drilling breaks were noticed, which is consistent with the GDR (page 34, Section 5.2.1): "Drilling breaks occurred frequently along foliation in almost every core run"; also, splitting along the foliation planes occurred during overcoring of the CSIRO HI cell by Agapito, see photo below (Arup Phase 2 Geotechnical Data Report FINAL, page 738). Finally, during the Oct. 26 site visit, the author was able to remove rock slabs isolated by foliation planes either by a gentle hammer tap or simply by hand; although these excavations were not smooth blasted, it is important that the damage was concentrated along the foliation. These four considerations indicate that foliation-parallel splitting will occur during blasting.

Figure B-17. HI Cell Run 4

10) How was D=0 carried over through the design specs and drawings, except for specifying a generic "smooth blasting"?
11) 3.4.4 and 3.4.5: In the analyses, a ubiquitous joint model was used to take into account the foliation; this model requires the cohesion, friction angle, and tensile cut-off along the foliation. In view of this model, direct shear tests should have been carried out on the intact foliation planes in order to obtain peak and residual strength values for cohesion and friction angle. No such a test was carried out. Direct shear tests were carried out on mechanically broken foliation planes (Phase 2 Site Investigation Geotechnical Data Report, page 26, Section 4.3.3), which provide no information on the peak strength parameters. Additionally, the tensile cut-off should have been determined by Brazilian tests with loading parallel to the foliation plane; for Brazilian tests, the GDR (page 46) does not provide the angle between loading and the foliation plane. The cohesion and friction angle (peak) along the foliation was determined in a convoluted and erroneous way:

a. Following Saroglou and Tsiambaos (2008), uniaxial and triaxial test data (irrespective of the type of schist!) were analyzed to obtain the variation of Hoek-Brown m_i parameter and the uniaxial compressive strength with the angle beta between the minor principal stress and the foliation.
b. Based on a selected value of beta, the peak cohesion and friction angle were calculated by linearizing the Hoek-Brown failure criterion over a selected range of confining stresses. However, these values for peak cohesion and friction angle are NOT the peak cohesion and friction angle along the selected foliation because the plane of failure is always inclined 45°+φ/2 on the direction of the minor principal stress.
c. The selection of beta angle could not be carried out on the basis of the back-analysis because the models in the back-analysis are insensitive to the angle beta, see Comment 4 on the PDR below.

12) 3.5.3 direct shear strength parameters were not distinguished for the 3 schists and there is no mention that the reported values refer to mechanically broken foliation planes, i.e. the reported values may refer to residual values.
13) 3.6.1 "Results show that the stiffness increases with decreasing angle. This may be a result of interlocking of vertically oriented foliation planes when they are subject to compression" This explanation reveals very poor understanding of basic rock mechanics: when loading parallel to the foliation, the deformability of the foliation plays no role on the Young's modulus parallel to loading.
14) 3.7 joint stiffness was calculated by using the rock mass modulus (obtained through correlation with GSI, and therefore for an isotropic material) and the joint spacing (no foliation, but 1 out of 4 joint sets); the used formula gives the deformation modulus across joint planes for an anisotropic rock mass. This is clearly incorrect.
15) 3.8, rock spring stiffness: calculated for the minimum width of the loaded foundation slab area. As a consequence, the stiffest value for the rock spring stiffness is calculated; this may not be conservative for a bedded beam model.
16) 3.9 rock mass dilation. Minimum confinement of 5 MPa was used; but this value is unattainable even by very thick cast in place linings for small tunnels. Also, why plastic shear strain of 0.3%, based on what?
17) 3.10 In situ stress. The "scatter, variability" etc. in this paragraph may actually depend on the solution used to analyzed the data. However, the design critically depends on the in situ stresses. In fact, principal stress directions are not at 90 deg in Fig. 27. Indeed, Golder 2010 report, page 6 "The calculations of stress from the overcoring strains use the Duncan-Fama and Pender (1980) solutions for homogenous, isotropic rock". As for Agapito, see Table B-1, page 733 of ARUP GDR, where CHILE deformation properties are given. Contrast these assumptions with the anisotropic (transversely isotropic) rock in overcoring pictures, pages 734-739 in ARUP GDR. Lack of funding (mentioned during the Oct. 28 meeting with the project team and internal reviewer) is not an excuse to use the isotropic solution instead of the proper anisotropic solution because:

a. Triaxial and uniaxial tests in different directions were already available (pages 46-48 of the GDR); they were sufficient to determine the 5 elastic parameters for this transversely isotropic rock at no additional cost.
b. In order to get even more accurate results, 3 uniaxial tests with moduli would have been needed to determine the 5 elastic parameters at each stress measurement location. A total of 3 x 4 = 12 test for a $2,500 total value should be warranted to determine the in situ state of stress (i.e. the boundary conditions in all of the subsequent analyses) for a $ 1 Billion facility concentrated in a very small rock volume (as compared to an elongated structure like a tunnel).

In the following picture, taken from (Amadei and Stephansson, Rock Stress and its Measurement, 1997), Prof. Derek Martin shows the effect of using the proper anisotropic rock characterization and solution in analyzing rock strain measurements. Notice that the ratio of maximum to minimum Young's modulus was equal to 2, whereas in this project the measured value was about 1.5 (ARUP GIR, page 33, Section 3.6.1).

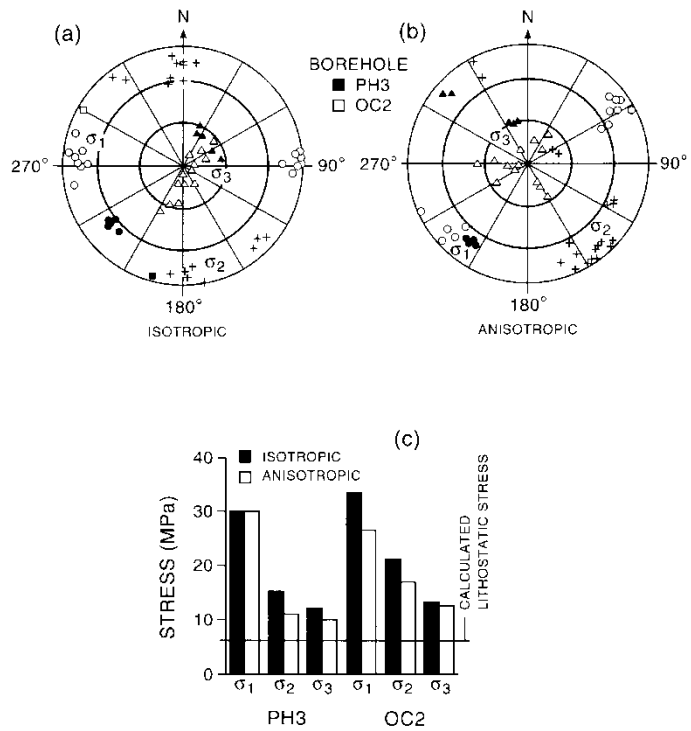


Fig. 5.35 Comparison of principal stresses obtained from overcore tests in boreholes PH3 and OC2 at the 240 level of the URL site. (a) Stress orientation assuming isotropic rock properties, (b) stress orientation assuming anisotropic rock properties, (c) comparison between stress magnitudes obtained with isotropic and anisotropic analyses. (After Martin and Simmons, 1993.)

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1) Page 35 and Appendix M (page 732 and following):
    a. "The Hoek-Brown strength criterion should be applied in areas dominated by contorted rock structure and kinematic block fall behavior, implementing the α=90° set of input parameters" Use an isotropic failure criterion to determine kinematic rock block failures?
    b. "The ubiquitous joint model should be applied in areas dominated by planar foliation subject to slabbing / buckling failures, implementing the α=90° set of input strength parameters for the rock mass and the reduced α=45° set of input strength parameters along the foliation plane". This approach and its limitations have been addressed in Item 11 of the "Comments on Arup 100% Preliminary GIR for LBNF" above.
2) 5.10.1 2D Distinct Element Method (UDEC): the ubiquitous joint model is used with foliation parallel to the excavation when foliation is actually at 60 deg to the cavern axis. This model should not be used at all.
3) 5.9 Kinematic Analysis. "Kinematic analyses have been performed to check whether the temporary ground support determined from the empirical method is sufficient to stabilize blocks". Why temporary support? Isn't it both temporary and permanent?? "The ground support installed for the LBNF project is considered permanent and will have a minimum 50 year design life" (page 40)
    a. Foliation is NOT included in the sets of "joints"; therefore, rock blocks formed by foliation planes are not included in the analyses. This is a major limitation because "Foliation is the dominant rock mass fabric of the Poorman Formation within the project area" (ARUP GDR, page 37)
    b. The center access drifts 1, 2, and 3 are subparallel to the foliation, and all cavern faces are subparallel to the foliation (both during construction and in their final stage). No buckling analysis was carried out when instead the existing 4850-18 EAST ACCESS DRIFT 1 shows major signs of buckling (see picture below taken during the site visit). The cavern faces that will advance toward the East will be driven against dip with the foliation plane will daylight at all times; this will create continuous face stability problems during construction.

4) Appendix M, Back analyses
   a. #6 Winze: **any** ubiquitous joint model is realistic, regardless of the shear strength along foliation

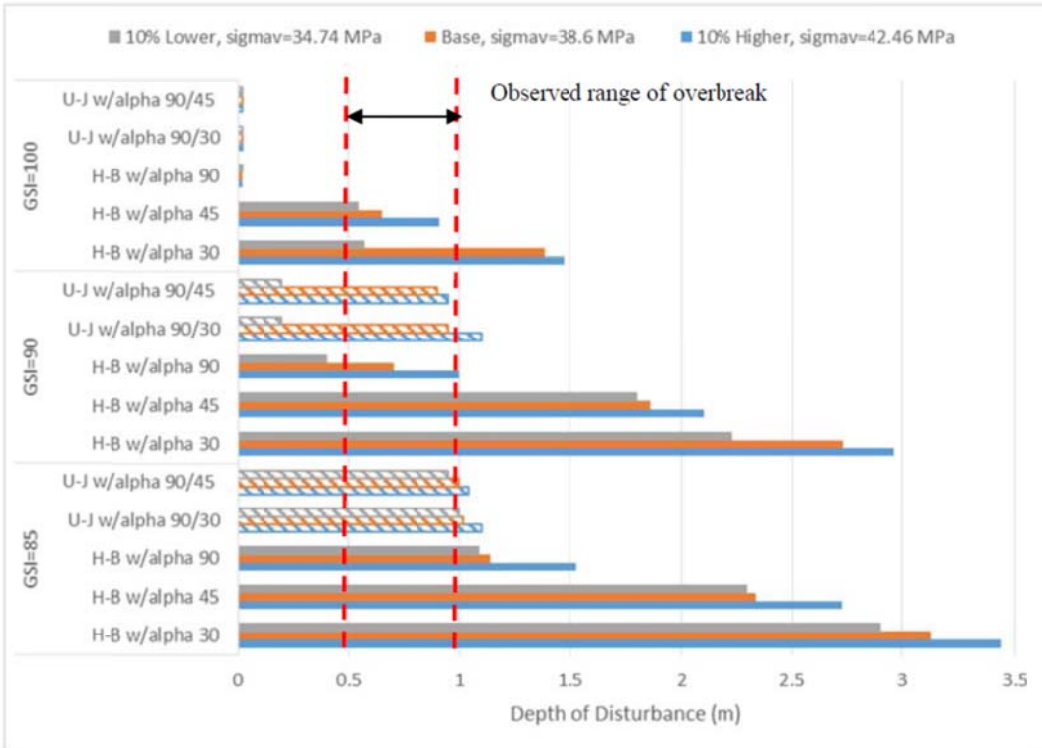


Figure 3.3: Depth of plasticity for various constitutive models and strength envelopes used in the #6 Winze Hoist Room backanalysis.

b. 3.3 Case Study 2: Governor's Corner Pillar
   i. "Midas GTS: 3D FE continuum based code which allowed quick importation of laser scan data to allow modeling of true intersection geometry. Doesn't include U-J model" not true, see below excerpt from the Midas GTS manual.

3.10 Jointed Rock Model

3.10.1 Introduction

Most soils and rocks, as a natural body, are anisotropic. That is, they have different properties in each direction. Anisotropy can be defined as a difference in a material's property such as elastic moduli when measured along different axes. Anisotropic materials are classified into two categories: elastic anisotropic and plastic anisotropic ones.

In this chapter, the Jointed Rock model is discussed as the anisotropic elastic perfectly-plastic model. The Jointed Rock model covers both the transversely isotropic elastic and the anisotropic plastic models. It is very useful to simulate the behaviors of stratified and jointed rock layers. There are several assumptions in the Jointed Rock model. This model can consider up to three directions of layers in an element. The element is defined as the transversely isotropic elastic material having five properties, which are defined in the orientation of the first joint, and one direction. The fault zone in the model is generally regarded as an anisotropic. The shear strength at interface is determined by the Mohr-Coulomb failure criterion. After the failure occurs, the plastic behavior governs the model. The different values of the shear strength or the material properties can be defined for three sliding planes.

ii. any H-B and ubiquitous joint model is realistic, regardless of the shear strength along foliation, see below

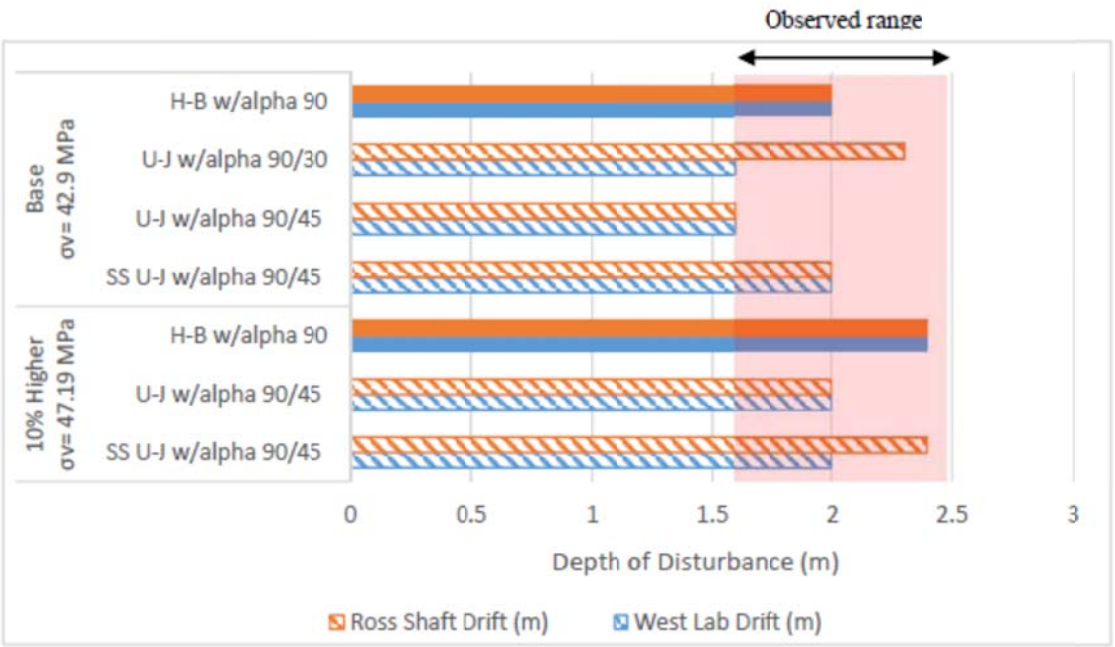


Figure 3.11: Depth of disturbance for various stress and strength envelopes used in FLAC3D.

3.4 Case Study 3: #4 Winze Drift. Observed wedge failure is obtained for alfa = 30 and alfa= 45. Reinforcement was not modelled and this may be the cause of the failure at lower GSI for alpha = 30.

In the UDEC model, wedge failure occurred along a persistent joint-vein and two cross-joints; it appears that the strength (cohesion and friction) of the ubiquitous model has little to do with the wedge failure on the left abutment; only tensile strength of the ubiquitous model can affect the left abutment. Cohesion and friction of the ubiquitous model may cause failure on the right abutment.

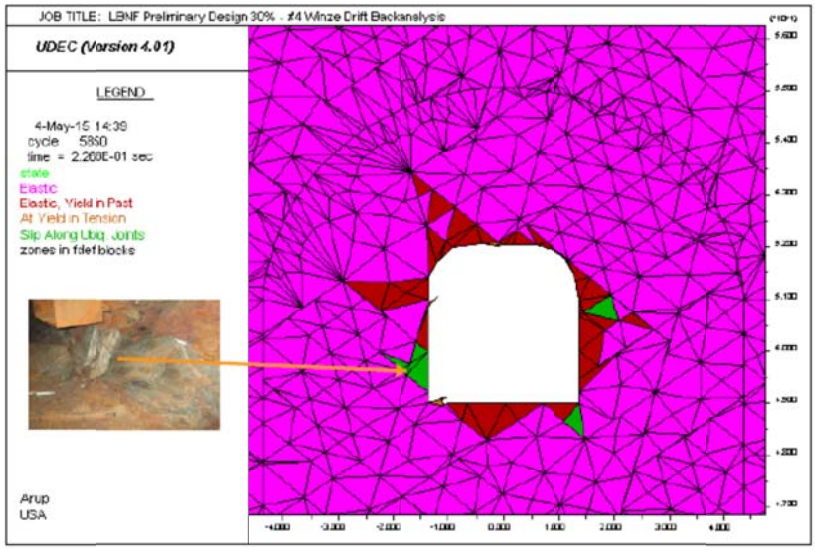


Figure 3.13: Yielding along ubiquitous joint plane in UDEC (GSI=85, α=45° derived strength for U-J plane).

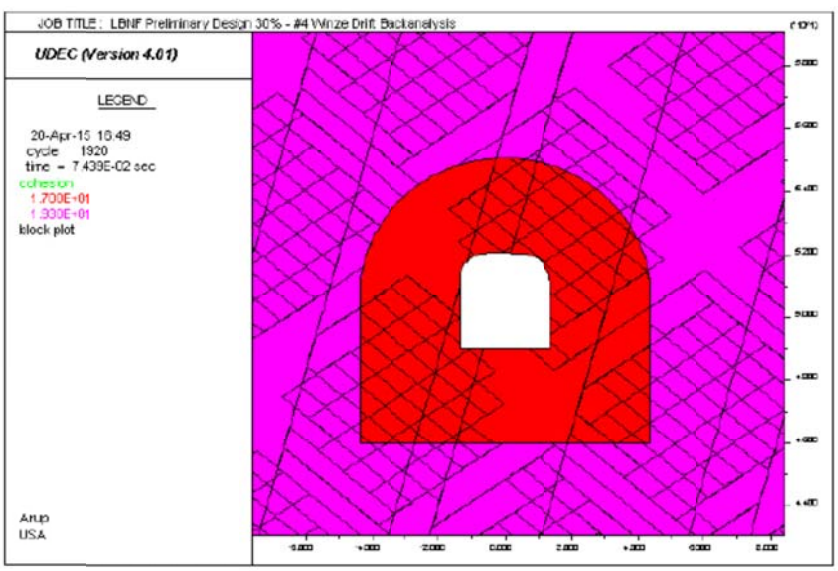


Figure 3.12: UDEC section (looking down the #4 Winze drift at 146°) used for backanalysis of slabbing failure mechanism. Two sets of M-C strength parameters are used in the model; associated with a confining stress of 20.7 MPa in the far field (purple) and 5 MPa (red) within 1 diameter of the adit boundary.

In conclusion, the back analysis that was carried out was inconclusive because the numerical analyses carried out indicate nothing about (are insensitive to) the strength parameters to be used for the foliation.

5) Appendix D, ESR = 1 was used (page 283). ESR = 1 is to be used for common civil engineering excavations, not for caverns that house for over 50 years a +650 million piece of equipment for a critical Physics experiment over a 3x150x20 = 9,000 m2 footprint. This raises the general question as to the safety level used for the entire excavation project: what is it? Has it been agreed upon with the physicists and all other stakeholders? See also comments below on bolt design strength. In addition, design criteria and codes used in the design are not stated in the report (as opposed to the 2015-08-24_BSI-100 Percent PDR).
6) Appendix H, page 5 (page 526 of pdf document): parameters for the foliation plane:
    a. Cohesion (U-J plane) MPa 5.11
    b. Friction Angle (U-J plane) Degree 42
    c. Dilation (U-J plane) Degree 22
    d. Tension (U-J plane) MPa 4.0 (no test measured the tensile strength across the foliation plane)

    used in the analyses are not in agreement with the GIR (page 32, table 3.9)

Table 3.9: Rock mass Mohr-Coulomb Properties, α =45° for Various Stresses

σ'_3 max	Range of Anticipated Values		Representative Value	
	Cohesion, C′	Friction Angle, ϕ'	Cohesion, C′	Friction Angle, ϕ'
3,000 psi (20.7 MPa)	680 - 2,180 psi (4.7 – 15.0 MPa)	29 - 38	1,390 psi (9.6 MPa)	36
725 psi (5 MPa)	390 - 1,970 psi (2.7 – 13.6 MPa)	40 - 45	1,120 psi (7.7 MPa)	44

7) Appendix H, page 41 (page 562 of pdf document): grout cohesive strength is completely unrealistic: 287 kN/m
    a. Perimeter of 32 mm bar => 2.86 MPa steel-mortar bond strength! Too high for a factored strength
    b. Perimeter of 65 mm hole (page 611 for CT-Bolt M33) => 1.4 MPa Bond strength! Too high for a factored strength, see tables below for ULTIMATE bond stress and resistance factors (AASHTO Bridge Specifications)

Presumptive average ultimate bond stress for ground/grout interface along anchor bond zone (after PTI, 1996).

Rock		Cohesive Soil		Cohesionless Soil	
Rock type	Average ultimate bond stress (MPa)	Anchor type	Average ultimate bond stress (MPa)	Anchor type	Average ultimate bond stress (MPa)
Granite and basalt	1.7 - 3.1	Gravity-grouted anchors (straight shaft)	0.03 - 0.07	Gravity-grouted anchors (straight shaft)	0.07 - 0.14
Dolomitic limestone	1.4 - 2.1	Pressure-grouted anchors (straight shaft)		Pressure-grouted anchors (straight shaft)	
Soft limestone	1.0 - 1.4	• Soft silty clay	0.03 - 0.07	• Fine-med. sand, med. dense – dense	0.08 - 0.38
Slates and hard shales	0.8 - 1.4	• Silty clay	0.03 - 0.07	• Med.–coarse sand (w/gravel), med. dense	0.11 - 0.66
Soft shales	0.2 - 0.8	• Stiff clay, med. to high plasticity	0.03 - 0.10	• Med.–coarse sand (w/gravel), dense - very dense	0.25 - 0.97
Sandstones	0.8 - 1.7	• Very stiff clay, med. to high plasticity	0.07 - 0.17	• Silty sands	0.17 - 0.41
Weathered Sandstones	0.7 - 0.8	• Stiff clay, med. plasticity	0.10 - 0.25	• Dense glacial till	0.30 - 0.52
Chalk	0.2 - 1.1	• Very stiff clay, med. plasticity	0.14 - 0.35	• Sandy gravel, med. dense-dense	0.21 - 1.38
Weathered Marl	0.15 - 0.25	• Very stiff sandy silt, med. plasticity	0.28 - 0.38	• Sandy gravel, dense-very dense	0.28 - 1.38
Concrete	1.4 - 2.8				

Note: Actual values for pressure-grouted anchors depend on the ability to develop pressures in each soil type.

Table 11.5.6-1 Resistance Factors for Permanent Retaining Walls.

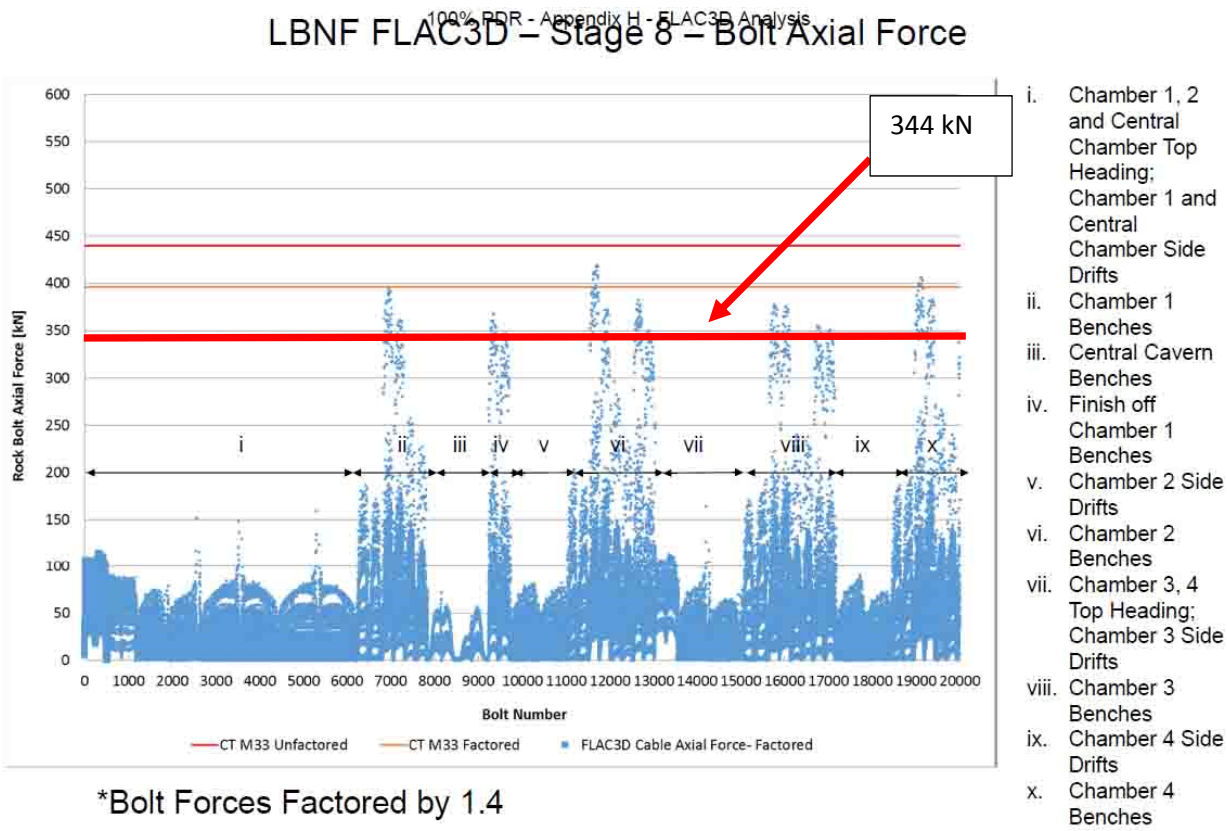
WALL-TYPE AND CONDITION		RESISTANCE FACTOR
Nongravity Cantilevered and Anchored Walls		
Bearing resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		1.00
Pullout resistance of anchors[1]	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65 [1] 0.70 [1] 0.50 [1]
Pullout resistance of anchors[2]	• Where proof tests are conducted	1.0[2]
Tensile resistance of anchor tendon	• Mild steel (e.g., ASTM A 615M bars) • High strength steel (e.g., ASTM A 722M bars)	0.90[3] 0.80[3]
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls		
Bearing resistance		Article 10.5 applies
Sliding		Article 10.5 applies
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements[4] • Static loading • Combined static/earthquake loading Grid reinforcements[4] [5] • Static loading • Combined static/earthquake loading	 0.75 1.00 0.65 0.85
Tensile resistance of geosynthetic reinforcement and connectors	• Static loading • Combined static/earthquake loading	0.90 1.20
Pullout resistance of tensile reinforcement	• Static loading • Combined static/earthquake loading	0.90 1.20
Prefabricated Modular Walls		
Bearing		Article 10.5 applies
Sliding		Article 10.5 applies
Passive resistance		Article 10.5 applies

[1] Apply to presumptive ultimate unit bond stresses for preliminary design only in Article C11.9.4.2.

[2] Apply where proof test are conducted to a load of 1.0 or greater times the factored design load on the anchor.

[3] Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to F_y. For high-strength steel apply the resistance factor to guaranteed ultimate tensile strength.

c. Loads on bolts (page 611 for CT-Bolt M33): yield load is 383 kN for a fully grouted bolt. Even by using a strength factor of 0.9 (see AASTHO table above), one gets a design load of 0.9 x 383 = 344 kN



By comparing this factored resistance with the calculated forces, bolts that do NOT check are in Chamber 1 Benches, Finish off Chamber 1 Benches, Chamber 2 Benches, Chamber 3 Benches, Chamber 4 Benches. The sentence “Factored rock bolt loads are also confirmed against the factored tensile strength and found to be acceptable” (page 39) is thus not true.

Additionally, the FLAC model appears NOT to take into account the pretensioning force of 10-15 kips (45-67 kN) specified in the drawings. Pretensioning would add to the loading on the bolts caused by rock mass displacements.

Bond between steel and grout: drawings specify CT M33 rock bolts; page 611 for CT-Bolt M33 shows a pvc sheathing around the steel, “use of plastic PVC corrugated tubing” (page 46), so there is NO bond between steel and grout. As discussed in the meeting with the design team and the internal reviewer, in the CT-Bolt M33 the bond stress is caused by the rebar deformations that dilate the bar-grout interface; this dilation causes an increase in normal stress, but the friction angle is limited by the presence of the PVC sheathing. As a consequence, the FLAC modeling of the reinforcement is wrong.

The presence of a PVC sheathing is also in contrast to the statement (page 46) "It is recommended that post-grouting occur in advance of the next excavation heading or at a maximum distance of 32 ft. (10 m) from the face, whichever is less, to inhibit excessive joint shear displacements." Because:

- There is no benefit to the rock mass by grouting a bolt where there is no bond btw steel and grout (see above), and
- The bolts are installed on a vertical plane, i.e. at 10-20° to the foliation plane and associated joint. They will cross very few foliation planes where they could contribute shear strength; when they intersect a foliation plane, the angle is very small, and the shear strength of the bolt will be hardly mobilized.

Actually, grouting the bolts right away will lead to cracking of the grout when the grout (in contact with the rock) will be put in tension by the subsequent stages of the excavation. Cracks will decrease the life of the bolt.

8) Appendix H, pages 29-31 (page 550-552 of pdf document) and pages 51-53 (page 572-574 of pdf document): excavations are surrounded by tensile stresses (in which direction?) of 2 MPa. How does this tensile stress affect the rock integrity (over the short and the long time) and the stability of the rock mass? If shotcrete is applied to the rock mass as excavation progresses downward, shotcrete will be subjected to the same 2 MPa tensile stress; this tensile stress will crack the shotcrete especially in its early stages of the curing process.
9) 5.11 Proposed Ground Support Systems. "minimal difference between the U-J and H-B models" because the strength of the foliation is simply unrealistic.
10) 5.11.1.2 Shotcrete. Structural checks on the shotcrete (page 358) use a model where the shotcrete is pinned-supported at the bolt heads:

Initial Shotcrete Lining Design Failure Mode Checks

INPUT PARAMETERS

γ = 27.6 kN/m^3

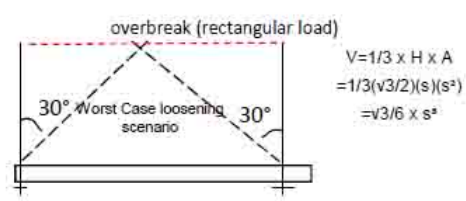
S [m]	W [kN] Prism	0.5m	1.0m
1.00	8.0	13.8	27.6
1.25	15.6	21.6	43.1
1.50	26.9	31.1	62.1
1.75	42.7	42.3	84.5
2.00	63.7	55.2	110.4

γ ..... Density of rock
S ..... Bolt spacing
W ..... Weight of block
0.5m, 1.0m .... Thickness of potential overbreak

APPLIED LOADING

1) Weight of block

$$W = \gamma \cdot \sqrt{3} \cdot \frac{S^3}{6}$$



However, Section 5.11.1.2, page 46, says “As the shotcrete is not explicitly included in the numerical analyses, the timing of placement is flexible and can be adjusted to suit the encountered ground conditions. It is anticipated that some rounds will require immediate support by shotcrete (flashcoat), especially those where the localized foliation trend aligns with that of the excavation. For aesthetic purposes, the shotcrete will cover all bolt head end hardware. Details are shown on the design drawings” This means that the contractor may install the shotcrete after the rock bolts (he’ll be incentivized to do that because he does not have to come back and cover the bolt heads), and therefore he’ll nullify the structural model => shotcrete may fail because it’ll be left without pin supports.

11) 5.11.3 Access Drifts and Mucking Ramps. The center access drifts 1, 2, and 3 are subparallel to the foliation. No buckling analysis done when instead the existing 4850-18 EAST ACCESS DRIFT 1 shows major signs of buckling, see picture below taken during the Oct. 26 site visit.

Instead, the report says "It is anticipated that initial support for the majority of the access drifts will only require spot bolts to stabilize localized rock blocks" (page 47); this approach will promote relaxation of the schists normal to the foliation plane, and extensive buckling of the thin rock layers between the foliation planes. Geol. David Vardiman reported at the 10/27 breakout session that spot bolting lead to rock failure and continuous maintenance needs in the 4850-18 EAST ACCESS DRIFT 1.

12) 6.5.6 Anticipated Blast Vibrations.
    a. The ground speeds depend on the charge weight; what is the charge weight used in the calculations?
    b. Limits are just given in terms of velocities, but the criteria must also take into account the frequency of the vibrations.
13) 6.5.7 Blasting Vibration Criteria:
    a. The ground speeds depend on the charge weight; what is the charge weight used in the calculations?
    b. "Blast vibration criterion of 0.50 in/sec (13mm/s), measured at the Ross Campus," where does this criterion come from?
14) 6.6 Air Blast Overpressure. Typical thresholds in unconfined environment are as follows:

    Pain threshold: 130 dB
    Hearing damage: 120 dB
    Jackhammer at 1 m: 100 dB

    "Air blast overpressure will be limited to 134 dB per Bureau of Mines recommendations to reduce the risk of injury to underground personnel". Is this overpressure at the blast holes or where personnel may be standing? In the latter case, this limit in a confined environment is too high.

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