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CHARACTERIZATION OF A WEAK ROCK MASS AND GEOENGINEERING ANALYSES FOR A CANYON LANDFILL IN NORTHERN CALIFORNIA

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ABSTRACT

A case history of extensive 60-m high excavations in a weak rock mass for a canyon landfill in Northern California is presented. The landfill is underlain by the Panoche Formation, a complex series of sandstones, siltstones, claystones, shales and conglomerates, thought to be dominantly turbidites, deposited as sub-sea fan deposits. As part of the design, kinematic analyses were performed, accompanied by two independent approaches used to evaluate the strength of the rock mass. One approach was based on laboratory rock core testing, while the second approach was based on the Hoek and Brown Criterion using mostly field observations. The two approaches yielded consistent results. Characterization of the rock mass indicated a pronounced improvement in the rock structure and the condition of the discontinuities with depth, resulting in an increasing Geologic Strength Index (GSI) with depth. Subsequent analyses performed using a layered Hoek and Brown Criterion allowed further steepening of rock excavations. Comparisons are also made in the results of the analyses using a layered vs. a more commonly used uniform Hoek and Brown approach. It was observed that the layered approach identified more critical, relatively shallow failure surfaces and eliminated the spurious apparently critical deep-seated rock mass failure surfaces, generated assuming a uniform rock mass.

INTRODUCTION

Strict regulations govern the design and construction of Municipal Solid Waste (MSW) landfills in the United States. The design of canyon landfills is also significantly affected by the site conditions, including the topography and the geologic conditions. Large excavations are required to maximize the airspace that can be used for waste placement. Engineering analyses need to consider the strength of the underlying subgrade materials, as well as the presence of landslides that may affect the stability of the excavated slopes.

A case history is presented of the successful increase in waste capacity by steepening the canyon excavation slopes in weak rock for the Vasco Road Landfill (VRL) located in Northern California.

SITE AND GEOLOGIC DESCRIPTION

The VRL is located in Alameda County, California, in the San Francisco Bay Area. The landfill is operated since 1963 and has a total permitted waste capacity of approximately 23 million cubic meters. The permitted landfill area is divided into disposal units. A new disposal unit is constructed as needed to provide required airspace for incoming waste.

The landfill is located within the northern Diablo Range, a sub-region of the northern California Coast Range Physiographic Province. The regional terrain is characterized by northwest-trending steep hills and narrow valleys, which conform to the overall topographic character of the San Francisco Bay Area.

Bedrock at the site is stratigraphically assigned to the Upper Cretaceous Panoche Formation, which is approximately 100 to 65-million years old. The Panoche Formation is approximately 7,300-m thick (Diblee and Darrow, 1981) and belongs to the Great Valley Sequence, the remains of a stack of sedimentary rocks deposited within a once seismically active, large marine forearc basin that developed in the late Mesozoic and early Cretaceous, adjacent to an ancient subduction zone (Dickinson and Seely, 1979). The Panoche is a complex series of sandstones, siltstones, claystones, shales and conglomerates thought to be dominantly turbidites deposited as sub-sea fan deposits. The two predominant sub-units of the Panoche formation at the VRL are:

- gray, weathered to tan or buff, fine- to medium-grained, massive or thickly to thinly bedded arkosic sandstones, with minor micaceous shale interbeds, often containing large concretions and moderately to highly prone to landsliding; and

- blue-gray, weathered to brown, argillaceous to silty, locally concretionary, micaceous clay shale, siltstones, and claystones with some thin sandstone strata and highly prone to landsliding.

PREVIOUS STUDIES AND RECOMMENDATIONS

For the construction of previous disposal units, extensive geologic mapping was performed to characterize the structure of the rock mass. The interpreted rock structure was used to perform kinematic analyses. In addition, drilling was performed to collect rock samples for laboratory testing. Triaxial compression tests and direct shear tests were performed on samples representative of the stronger sandstone and the generally weaker siltstone/claystone. The shear strength used in the analyses was expressed in terms of Mohr-Coulomb parameters and was primarily based on the direct shear test results performed along the discontinuities of the siltstone/claystone and the sandstone. Figure 1 illustrates results of the direct shear tests for sandstone, siltstone and claystone. The sandstone test results are in the upper bound of the data, whereas the siltstone test results fall in the lower range of the data, with the claystone data exhibiting significantly more scatter and strengths that are similar to the siltstone strengths or significantly higher. Triaxial compression test results on siltstone specimens also resulted in an average cohesion of 157 kPa and a friction angle of 33 degrees.

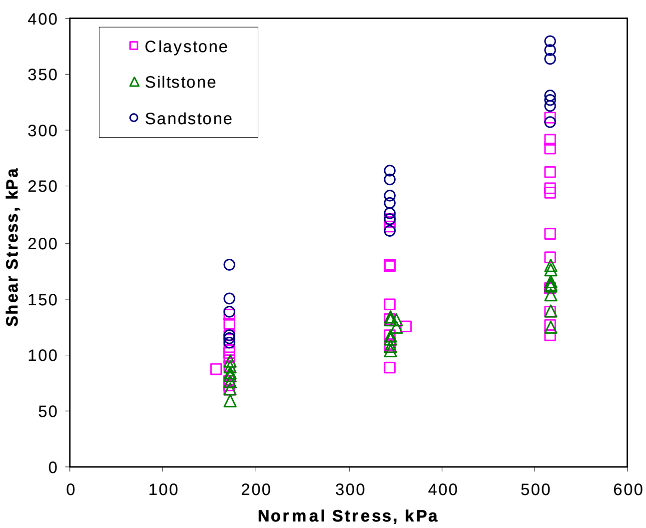

Figure 1: Direct shear data for claystone, siltstone and sandstone of the Panoche Formation.

Previous evaluations of the stability of the excavations were based primarily on the strength of the rock mass evaluated from the direct shear data, accounting for the fact that the rock mass will consist of a mixture of siltstone, claystone and sandstone. The results of the stability analyses indicated that the slopes were limited to generally 4H:1V (horizontal:vertical) at lower elevations, 3H:1V at higher elevations above the lined landfill areas, whereas the upper few meters could be as steep as 2.5H:1V. Benches were typically constructed every 15 m vertically. To improve stability, the construction of a network of drains that would lower the groundwater table about 1.5 m below the base grades was also recommended.

APPROACH

At the outset of the study summarized here, it was evident that some adjustments to the geotechnical engineering recommendations should be investigated. The geologic investigation for the design of Disposal Unit 9 (DU-9) Phase 2, performed in early 2006, included the evaluation of existing borehole logging and laboratory data and the observation of test pits excavated to a maximum depth of 5 m. The test pits were intended to reveal the presence of suspected shallow landslides, identify the depth to stronger rock, and be used to collect discontinuity attitudes to evaluate the rock mass structure.

The geologic investigation for the design of Disposal Unit 10 (DU-10), performed in late 2006 - early 2007, augmented the previous investigation by performing 4 boreholes to an approximate depth of 25 m. This paper presents the data collected and analyses performed as part of both designs.

The evaluation of the stability of the canyon slopes included two considerations that required separate engineering analyses:

a) consideration of instability governed by the rock structure, i.e., the formation of unstable planes or wedges along the predominant bedding and joints; and
b) consideration of rock mass stability, i.e., the evaluation of the slope stability for the average rock mass strength.

Rock Structure and Kinematic Analyses

Rock discontinuity attitudes (strike/dip) were collected from test pits and were supplemented by attitudes collected previously in adjacent landfill areas. A total of 64 bedding and joint attitudes were collected and were supplemented by the additional 138 attitudes collected previously. The data were projected to an equal-angle, lower hemisphere projection (Figure 2) and were statistically analyzed to develop Fisher concentration diagrams and identify the major bedding and joint discontinuities (Table 1). A Fisher concentration of 4% or higher was used to define the major discontinuities. One major bedding discontinuity and 4 major joint discontinuities were identified from these analyses.

Kinematic analyses, as described by Hoek and Bray (1981) and Goodman (1989), using all identified major discontinuities were performed to develop the maximum, kinematically allowable, slope inclinations for different orientations of cut slopes. The software program DIPS

(Rocscience, 2002) was used for the plane and wedge stability analyses. Because of the overall southwesterly bedding dips and prevailing joints, slopes facing from northwest to east were kinematically allowable to be as steep as 2H:1V whereas slopes facing southeast to west could be as steep as 3H:1V.

Table 1: Attitudes of major discontinuities.

	Strike (degrees)[1]	**Dip (degrees)**
Bedding	119	36
Joint #1	177	35
Joint #2	014	57
Joint #3	264	88
Joint #4	359	87

[1] *According to the right hand rule.*

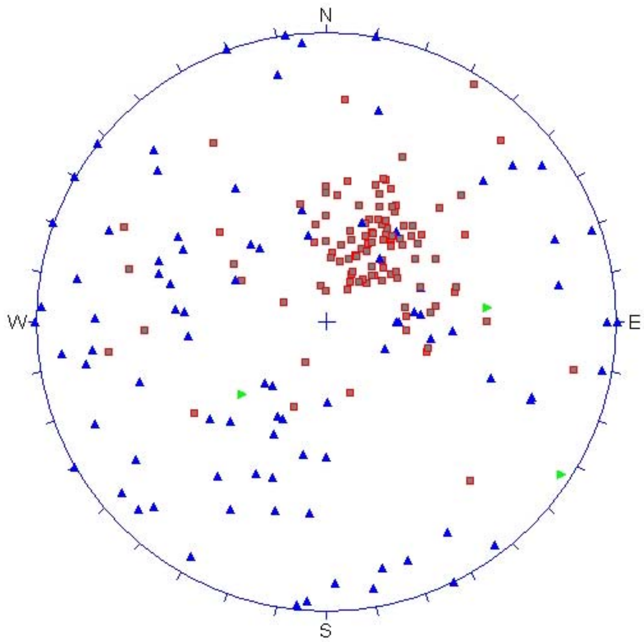

Figure 2: Equal-angle, lower hemisphere projection of collected discontinuity attitudes
(squares for bedding, triangles for joints).

Rock Mass Stability

Based on the test pit observations (Figure 3) and the borehole logging information, the rock mass was characterized by the Hoek and Brown Criterion (Hoek and Brown, 1980; Hoek et al., 2002). The Hoek and Brown Criterion provides estimates of the strength of jointed rock masses based on the assessment of interlocking rock blocks and the geomechanical condition of the surfaces between the blocks (Hoek and Karzulovic, 2000).

The input parameters necessary to develop the Hoek and Brown envelope are: the Uniaxial Compressive Strength of the intact rock σ_{ci}, the Geologic Strength Index GSI, the m_i material parameter and the disturbance factor D. Using available information, a constant value was assigned for all parameters to characterize the rock mass. Thus, one rock mass shear strength profile was used in these analyses. The selected input parameters are listed in Table 2. A value of 9,576 kPa (200 ksf) was used based on field estimates of strength and the guidance table provided by Hoek (2007). A conservative GSI value of 30 was used to characterize the rock mass even though, based on the existing boreholes, it was recognized that the GSI value likely increased with depth. A constant material parameter m_i of 4 was used based on the guidance provided by Hoek (2007) for a claystone material. Finally, a disturbance factor D of 0.7 was used for slopes excavated using mechanical methods. It was recognized that these values were generally representative and to some degree conservative, however, this level of conservatism was deemed necessary, since no deep subsurface investigation was performed, and only data available from previous studies was used.

Figure 3: View of the siltstone rock mass in a test pit at shallow depths (scale is 1 ft=30.5 cm)

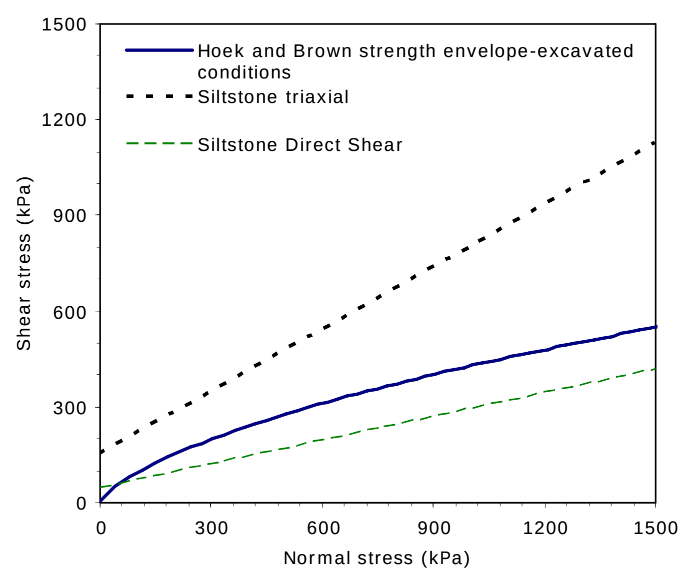

Figure 4: Hoek and Brown uniform and conservative envelope used in analyses for disposal unit DU-9 Phase 2 and comparison with previous existing laboratory data at the site.

A comparison of the strength envelope that was used in the analyses to the existing data is shown in Figure 4. The Hoek and Brown envelope is generally higher than the direct shear test results for siltstone, but lower than the triaxial compression test results. On the basis of these analyses, 3H:1V excavation slopes were recommended for DU-9 Phase 2, while the network of drains previously recommended was deemed unnecessary. The DU-9 Phase 2 excavation was successfully performed in summer of 2006.

Table 2: Uniform Hoek and Brown strength envelope

Input Parameter	Value
Uniaxial Compressive Strength of Intact Rock, σ_{ci}	9576 kPa
Geologic Strength Index, GSI	30
Material Parameter, m_i	4
Disturbance factor, D	0.7

DEEP SUBSURFACE INVESTIGATION PROGRAM

As part of the design of DU-10, deep subsurface investigation was performed. A total of 4 boreholes were performed to depths of 25m using HQ size double and triple-tubed core barrels to secure continuous rock core with a diamond rock bit. Piezometers were also installed in two of the boreholes to enhance the existing groundwater elevation data. Careful rock core logging was performed using the guidelines presented in the Engineering Field Geology Manual (Bureau of Reclamation, 1987).

All four boreholes generally revealed the presence of near surface, stiff colluvium soil, grading to intensely weathered or decomposed rock, which were generally not thicker than 2.4 m on the slopes. The investigation also revealed the predominance of siltstone, with isolated concretionary sandstone blocks. These observations were generally consistent with previous studies and the expected geologic nature of turbidites.

Figure 5: View of the core at a depth of approximately 25 m.

The rock mass structure varied from “disintegrated” near the surface to “intact” and “blocky” at higher depths (Figure 5). A pronounced improvement of the rock quality with depth was observed. The GSI profile was developed from the rock core and was documented for each borehole, similar to the recommendation of Hoek et al. (2005). An example is shown in Figure 6 for one borehole. The GSI value increases from values of about 10 to 20 near the surface, to values of about 60-80 at depths higher than 20 m. The GSI values systematically increase with depth for all 4 boreholes. Based on this information, a varying GSI with depth was selected for the design as shown in Figure 7. In comparison to the constant GSI=30 value used in DU-9 Phase 2, the new data indicated that lower GSI values were more representative at shallow depths, but significantly higher values are representative of the conditions at higher depths. Thus, the selection of a constant GSI value of 30 for analyses considering deep global instability, as adapted in the previous analyses, was in retrospect conservative.

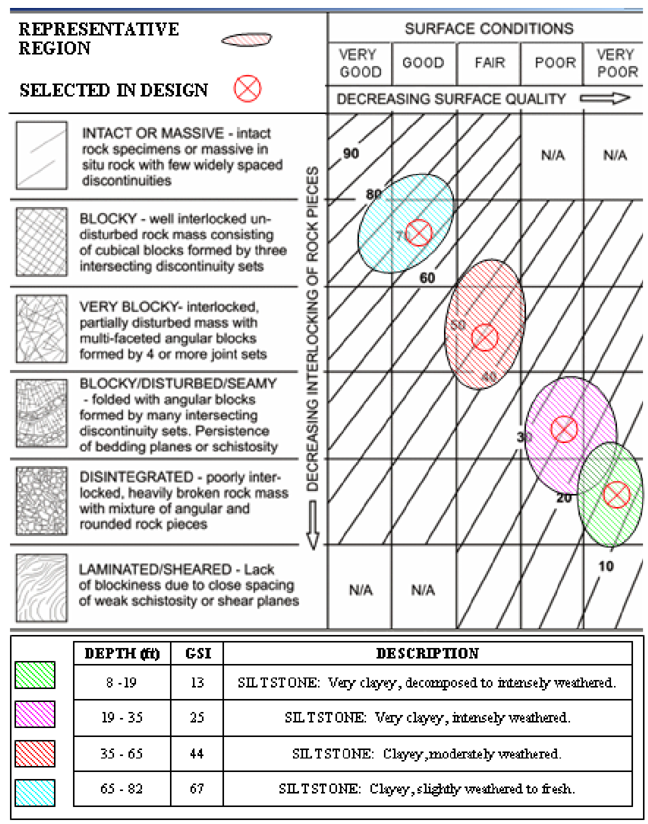

Figure 6: Variation of GSI with depth for a borehole.

Unconfined compression tests were also performed, but the specimens failed along pre-existing discontinuities inherent to the rock structure. Thus, the laboratory assessment of the uniaxial compressive strength of the intact rock was not possible. This is likely indicative of weak, highly fractured rock, where failure is encouraged to occur along some pre-existing discontinuity. However, field observations of the core, i.e., resistance of the core to hammer blows and knife incisions, suggested that the uniaxial compressive strength of the intact rock used in previous analyses, was still a reasonable value. The remaining Hoek and Brown input parameters were also the same. A constant disturbance factor D of 0.7 was used for the entire rock mass, even though it was recognized that the rock mass is likely more relaxed and weakened near the surface than at higher depths. However, in the absence of more specific recommendations, this level of conservatism was deemed prudent in design.

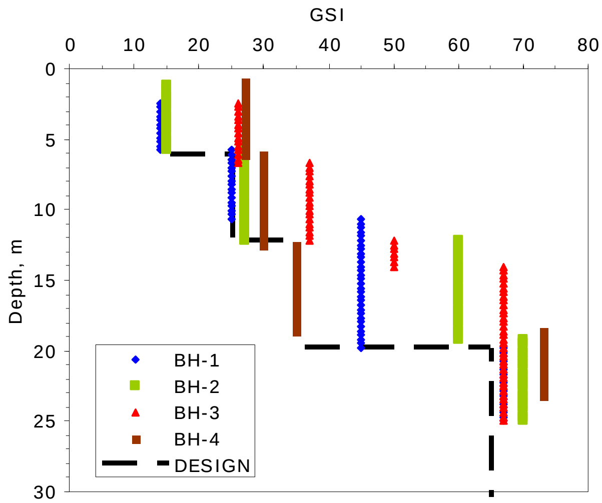

Figure 7: GSI vs. depth for all boreholes and the design.

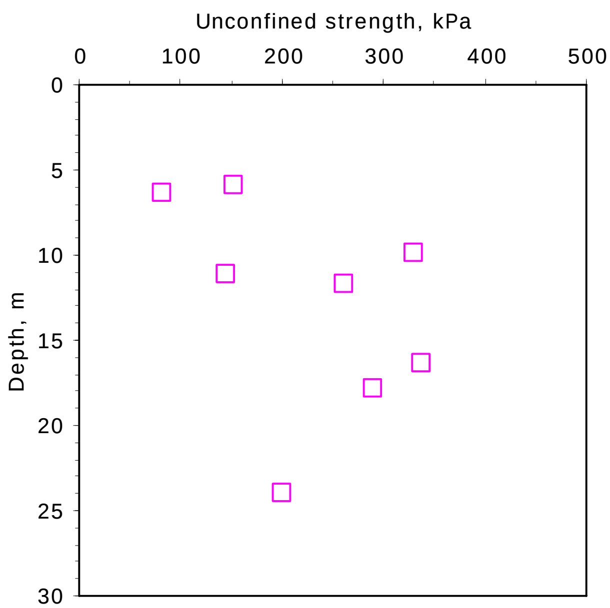

Figure 8: Unconfined strength of claystone/siltstone vs. depth

Samples from the core were also selected for laboratory testing. Unconfined compression tests were performed and the unconfined strength was generally found to increase with depth, as shown in Figure 8. As mentioned previously, all unconfined test specimens failed along discontinuities. Point load tests were also performed to obtain an index of the rock strength, even though the test is not generally recommended for weak rock masses. As shown in Figure 9, the unconfined strength increases with the Point Load Index I_s, however the increase in strength was generally lower than the increase predicted by the Bieniawski (1974) equation, which was developed for stronger rocks. It must be noted that the Point Load Index was not used in design, but only to compare the specimens tested in unconfined compression to the specimens tested in consolidated undrained triaxial compression.

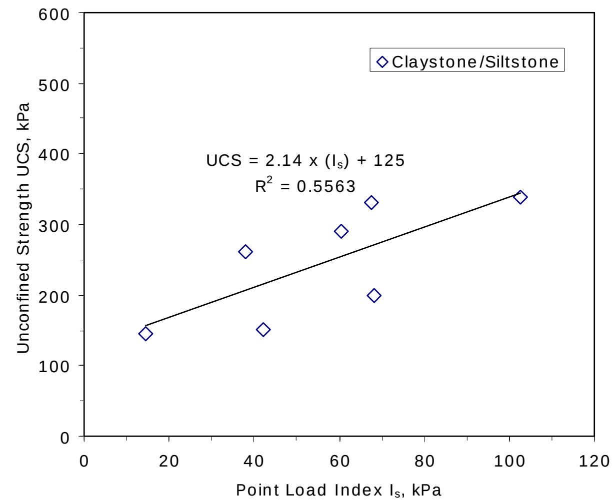

Figure 9: Unconfined strength of claystone/siltstone vs. point load index.

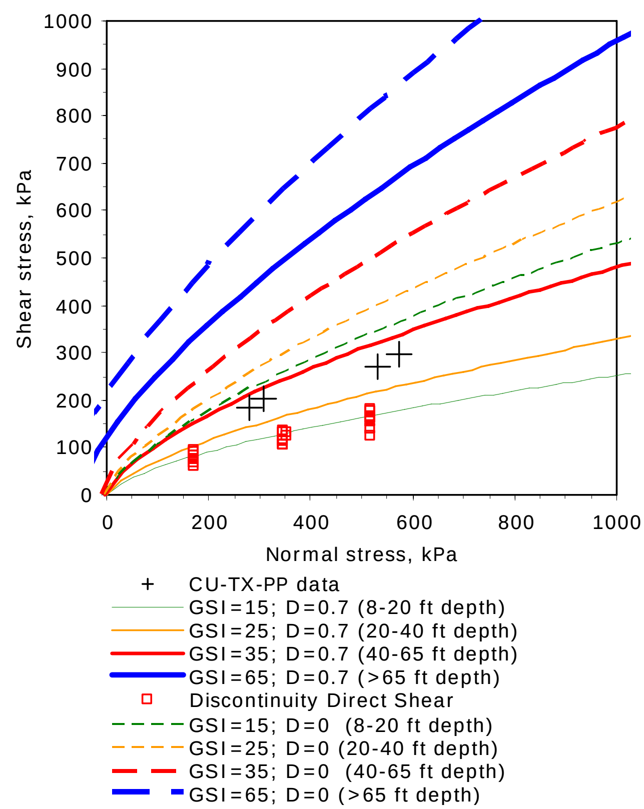

Figure 10: Hoek and Brown strength envelopes and comparison with the laboratory results.

Consolidated undrained triaxial tests with pore pressure measurements (CU-TX-PP) were also performed on two specimens from a depth of about 6.9 m and 17.8 m. The results of the tests were similar for the two specimens. The triaxial compression test specimens also failed along pre-existing discontinuities.

A comparison of the strength envelopes from the laboratory tests with that from the independent field-based Hoek and Brown approach is shown in Figure 10. Hoek and Brown envelopes are shown for each of the layers considered for undisturbed (D=0) and excavated (D=0.7) conditions. The lowest Hoek and Brown envelope is generally in the vicinity of the siltstone direct shear test results. The CU-TX-PP test results fall between the envelopes developed for GSI=25 and the GSI=35, which are representative of depths of 6.1-12.2 m and 12.2-19.8 m. Since the specimens were retrieved from depths of 6.9 m and 17.8 m, the CU-TX-PP results are consistent with the Hoek and Brown envelopes and provided an independent verification of the envelopes developed for the analyses.

STABILITY ANALYSES

Based on the findings of the field subsurface investigation the subsurface was divided into layers parallel to the original topography. The layers included (top to bottom): a 2.4-m thick layer of colluvium; a 3.7-m thick layer of rock mass with a GSI of 15; a 6.1-m thick layer of GSI=25 material; a 7.6-m thick layer of GSI=35 material; and a basal GSI=60 rock, which was assumed to extend to high depth. The remaining rock material parameters were the same for all layers and as listed in Table 2. Analyses were performed using the Slope/w program (Geo-Slope International, 2004) and Spencer's method of analysis.

An example cross-section for excavated conditions is shown in Figure 11. The layered approach accounts for the progressive increase in rock mass strength with depth. As a result, the layered model restricts the critical failure surfaces to the upper, weaker layers only, as opposed to the analyses performed for DU- 9 Phase 2 where large, deep-seated failure surfaces governed the stability of the cut slopes. The layered Hoek and Brown approach was considered more representative of the site conditions and a further increase of the recommended cut slopes was made in the design of DU-10. In areas allowed by the kinematic analyses, 3H:1V cut slopes were proposed for the sections of the slopes to be lined, with 2H:1V slopes for the sections of the slopes outside the lined areas.

The layered GSI Hoek and Brown approaches provided significant benefits compared to the uniform GSI approach originally used as part of the design of DH-9 Phase 2. Figure 12 illustrates the same cross-section, (a) modeled as a uniform rock mass (according to the approach used in DU-9 Phase 2) and (b) modeled using the layered approach (according to the approach used in the design of DU-10. The uniform Hoek and Brown approach resulted in relatively deep, critical (as deep as 50 m) failure surfaces with a minimum factor of safety of 1.4 (Figure 12a). Analyses using layered approach resulted in higher factors of safety. The critical failure surface (i.e., the failure surface with the minimum factor of safety) is a shallow (up to approximately 6 m deep) failure surface in the upper, weaker layers and has a static factor of safety of 1.5. This failure surface is not captured by the uniform Hoek and Brown material. In addition, using the layered approach, deep failure surfaces are not critical anymore since they extend to stronger rock masses. The critical failure surface using the uniform model has a factor of safety equal to 3.0 for the layered model.

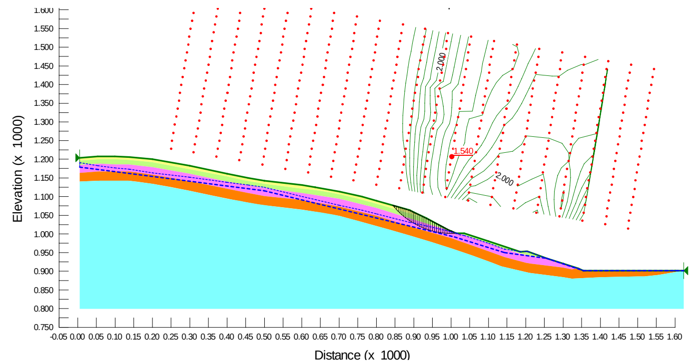

Figure 11: Analyses performed using a layered Hoek and Brown approach.

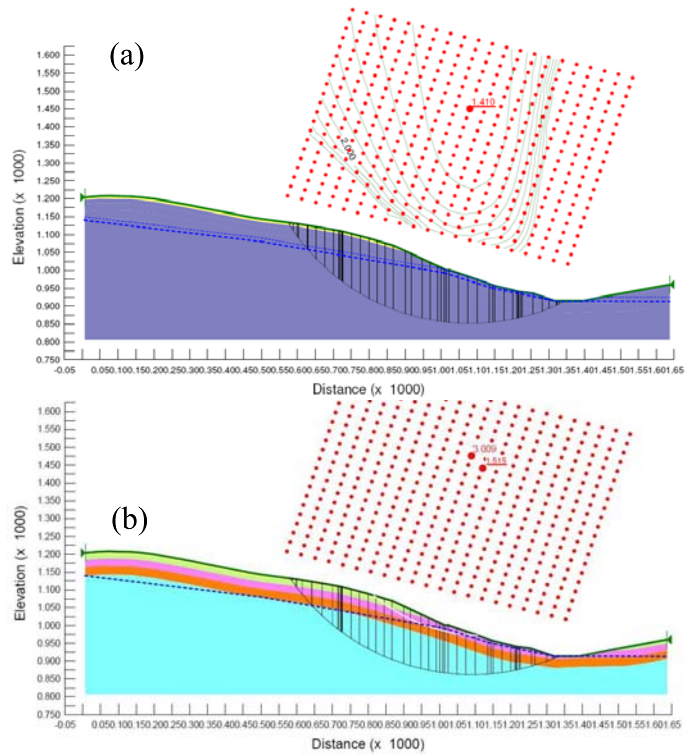

Figure 12: Comparison of the critical factors of safety for (a) the uniform Hoek and Brown material; (b) the layered Hoek and Brown material (critical failure surface is shown in white color) (lengths in ft).

DISCUSSION

Two independent approaches were used to evaluate the rock mass strength: (a) rock laboratory testing, and (b) the Hoek and Brown approach based on careful field observations of existing excavations and the rock core. The consistency in results between the two approaches resulted in an increased confidence in the geologic assessment of the rock mass.

The triaxial compression test results appear to be generally consistent with the derived Hoek and Brown envelopes. This may be attributed to the fact that the small-scale triaxial specimens of weak rock were representative of the average rock mass in the field. However, this may not be the case for stronger rocks. For intact specimens, higher strengths, representative of the intact rock may be estimated, whereas for fractured specimens, overly weaker results may be estimated if failure occurs along a predominant joint that is continuous at the specimen scale, but was not continuous at the slope scale.

The use of direct shear test results in the rock mass stability analyses was proven to be overly conservative. However, the direct shear test results were used in the kinematic analyses to evaluate the stability of planes or wedges sliding along existing discontinuities. In addition, a rock strength envelope based on the unconfined compression results (Figure 7) also resulted in significantly lower strengths than those estimated from Hoek and Brown and the laboratory data.

An important issue related to the use of the Hoek and Brown envelope is the use of the disturbance factor D. Whereas recommendations are provided by Hoek (2007) based on the excavation method and level of relaxation, it is reasonable to expect that the value of the disturbance factor should depend on the amount of excavation and the depth of the rock mass from the excavated face. In the absence of more detailed recommendation, the authors used a constant D value.

CONCLUSIONS

A case history of rock mass excavations on soft rock of the Panoche Formation is presented. Analyses were performed to evaluate the stability of the excavations against unstable wedges or planes along existing joints or the bedding, as well as the stability of the rock mass. In the evaluation of the stability of the rock mass, the strength of the rock mass is critical. Two independent approaches, one based on laboratory testing results and one based on the Hoek and Brown approach using field observations were employed and yielded consistent results. Originally, a uniform Hoek and Brown criterion was used to characterize the material. Subsequent investigation, indicated a pronounced increase in the strength of the GSI with depth. To accommodate this observation, a layered approach using the Hoek and Brown criterion was used, that allowed the succesful steepening of the rock excavations. The use of the independent approach to evaluate the strength of the rock mass allowed an increased confidence in the estimates of the input parameters.

NOTE

More information relevant to this paper is presented in the Geoengineer website at: http://www.geoengineer.org

REFERENCES

Bieniawski, Z. T. [1974]. "Geomechanics Classification of Rock Masses and its Application to Tunneling". *Proceedings of the 3rd International Congress for Rock Mechanics, International Society for Rock Mechanics*, Vol. IIA, Denver, 1974, pp. 27-32.

Bureau of Reclamation [1987]. "*Engineering Geology Field Manual*". U.S. Dept. of Interior, 599pp.

Dibblee, Jr., T.R., and Darrow. R.L. [1981]. "Geology of the Northern Diablo Range and Livermore Valley Area." Geology of the Central and Northern Diablo Range, Field Trip Guidebook for Annual Meeting Pacific Section SEPM, Field Trips 2 and 5. Ed. V. Frizzell. Society of Economic Paleontologists and Mineralogists. Los Angeles, CA. 29 May 1981.

Dickinson, W.R. and Seely, D.R. [1979]. "Structure and stratigraphy of forearc regions". American Association of Petroleum Geologists Bulletin. Vol. 63, No. 1, 1979: 2-31.

Geo-Slope International. [2004]. "SLOPE/W version 6, software for analyzing slope stability", part of the GEO-STUDIO 2004 suite of applications. Calgary, Alberta.

Goodman, R.E. [1989]. "Rock Mechanics". New York: John Wiley and Sons, 562 pp.

Hoek, E. [2007]. "Practical Rock Engineering, – An Ongoing Set of Notes.", http://www.rocscience.com.

Hoek, E. and Bray, J.W. [1977]. "Rock Slope Engineering". Revised 2nd Edition, London: Institution of Mining and Metallurgy. pp. 402.

Hoek, E. and Brown, E.T. [1980]. "Empirical strength criterion for rock masses". J. Geotech. Engng. Div., ASCE 106 (GT9). pp. 1013-1035.

Hoek, E., Carranza-Torres, C.T. and Corkum. B. [2002]. "Hoek-Brown failure criterion – 2002 edition". *Proc., North American Rock Mechanics Society*. Toronto, Canada. July '02.

Hoek, E., Marinos, P.G. and Marinos, V.P [2005]. "Characterisation and engineering properties of tectonically undisturbed but lithologically varied sedimentary rock masses". International Journal of Rock Mechanics and Mining Sciences. No. 42 (2005). pp.277-285.

Hoek, E. and Karzulovic, A. [2000]. "Rock-Mass properties for surface mines". Slope Stability in Surface Mining. Ed. W.A Hustrulid,. M.K McCarter, and D.J.A. Van Zyl. Colorado: Society for Mining, Metallurgy and Exploration, 2000. pp.59-69.

Rocscience. [2002]. "DIPS User's Manual". Version 5.1, Toronto, Canada.