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Dip Slope Characterization for Residential Development in Southern California using the Hoek-Brown Failure Criterion

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ABSTRACT: Typical practice within Southern California requires that while estimating the slope stability of dip slopes in weak, bedded rock, one of two toe-failure mechanisms be considered: (1) shearing at the toe along a prominent joint or (2) shearing at the toe through intact rock. Both toe failure mechanisms may be valid depending on the geologic conditions encountered at the toe of the slope. It is generally agreed that failure/shearing along the upper portion of the slope will occur along bedding. Slope stability evaluations are carried out by applying anisotropic strength relationships and the Mohr-Coulomb failure criterion. This paper presents a case history where the two mechanisms typically used to model failure through the toe did not apply because of the combined influence of non-persistent geologic structures and intact rock strength (i.e. rock mass strength). Instead, it was decided to use the Hoek-Brown Failure Criterion to estimate the shear strength of the rock mass at the toe of the dip slopes. This paper illustrates the application of the Hoek-Brown Failure Criterion to one geologic environment that was not conducive to the typical failure mechanisms experienced in Southern California. The results of this study suggest that given the proper geologic conditions, the Hoek-Brown Failure Criterion is appropriate to estimate the strength of a bedded rock mass where dip slope failures may be expected.

1. INTRODUCTION

High-density residential development within Southern California has becoming increasingly difficult for a number of reasons. The geology of the area is tectonically disturbed (folded and faulted), and seismically, is very active. Reviewing agencies are becoming more and more stringent in their requirements for grading permits because of the public's willingness to bring legal action. Yet the publics' demand for residential development motivates developers to maximize the amount of developed land above steep slopes.

A residential development consisting of 375 single-family residential lots has been proposed on the 230-acre site above Devil Canyon, north of the City of Chatsworth, California. The general location of the proposed development is shown in Fig. 1. Developments such as these are not unique. Because the geology of the area consists of some of the oldest and most highly indurated rock encountered within Southern California, the typical procedures employed to characterize rock masses for the purposes of evaluating slope stability may not be appropriate

Given the value of the land, the most challenging aspect of the geotechnical design was providing recommendations for the minimum setback distances along the slope crests above Devil Canyon; i.e. how close to the slope crests may development proceed, balancing issues of public safety and maximizing land use.

The south side of Devil Canyon consists of shallow-dipping slopes that are coincident with bedding and reach heights of 60 m. The north side of the canyon is steeper, with bedding dipping into the slope and heights approaching 85 m. At the base of the canyon is an intermittent stream. For the purposes of discussion the southern slopes will be referred to as "dip slopes". The north slopes are not coincident with bedding and are referred to as "anti-dip slopes".

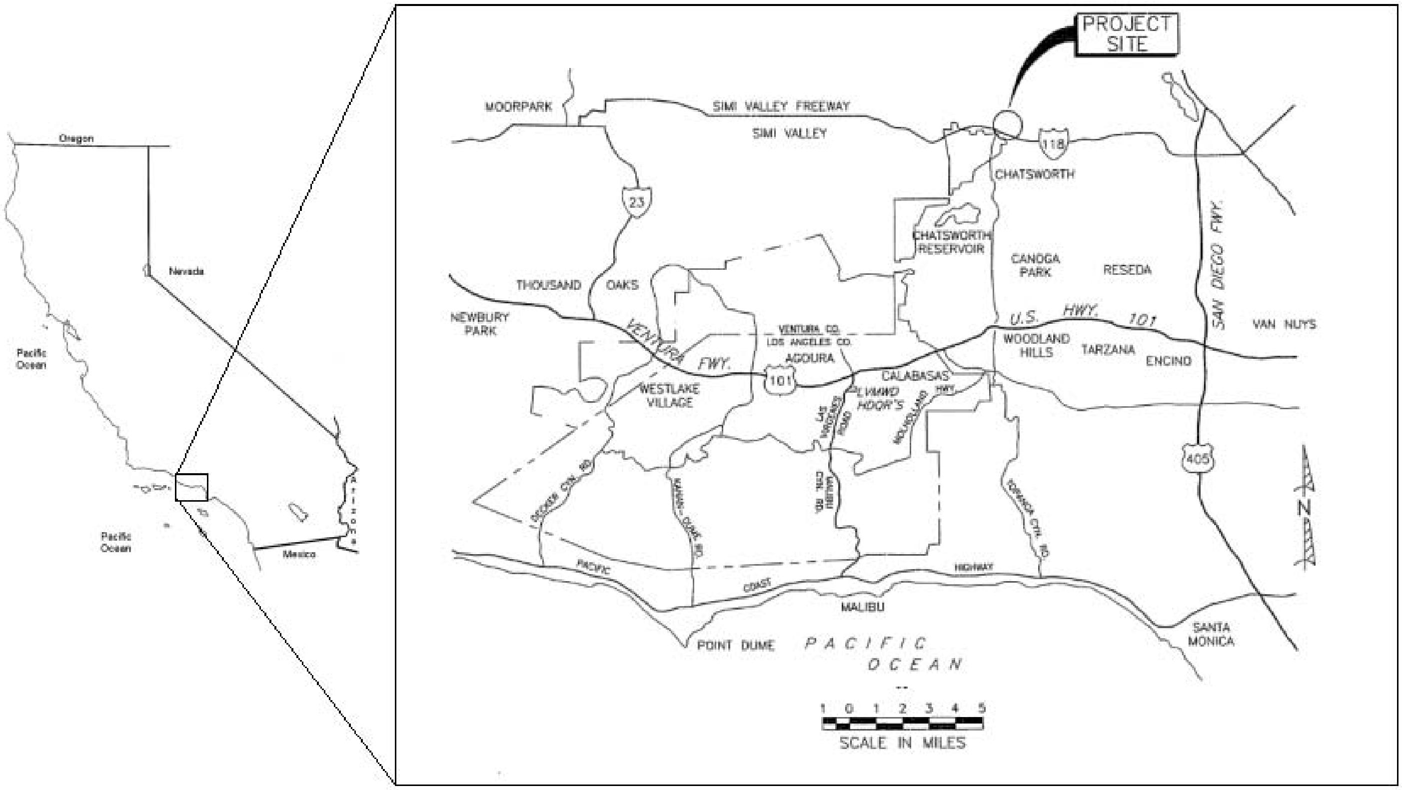

Fig. 1. Project Location Map

This paper focuses on and presents the results from the rock mass strength characterization and slope stability analyses performed to aid geotechnical recommendations for structural set backs on slopes coincident with bedding. The slope stability of the dip slopes was considered to be a function of the strength of the bedding planes and the rock mass at the slope's toe. A general consensus was reached between all interested parties regarding the strength model to be used for that portion of the failure plane forming along bedding. Considerable efforts, however, were required to reach a similar consensus regarding the methods undertaken to characterize the strength of the toe rock mass. This paper will also focus on this issue and summarize the different models proposed to model the rock mass at the toe of the dip slopes.

2. GEOLOGY OF THE PROJECT AREA

In the Devil Canyon area, the geology consists of slightly folded Cretaceous bedrock of the Chatsworth Formation. The type section for the Chatsworth Formation [1] is located along Woolsey Canyon road, a few miles south of the project site. The Chatsworth Formation was originally named and thought to be part of the Chico Formation [1,2]. It has been described as a turbidite series which generally consists of sandstone that is thickly-bedded and at least 1830 m thick [1]. At close proximity to the study area, about 80 percent of the Chatsworth Formation consists of thick-bedded sandstone with the remaining 20 percent consisting of thin-bedded, fine-grained sandstone and siltstone [2]. The thick-bedded sandstone series averages about 7 m with sandstone to siltstone ratio of about 20:1. The thinly bedded fine-grained sandstone and siltstone series averages 3 m with sandstone to siltstone ratios of 1:1. A geologic map of Devil Canyon is presented in Fig. 2.

2.1. Geologic Control on Failure Mechanisms

Thousands of discontinuity orientation data were collected during outcrop mapping, core logging and down hole televiewing. In general, the geological structure is similar throughout Devil Canyon. The structure is homoclinal. The general relationship of the orientations of the discontinuities within the rock mass are represented in Fig. 3. The data set represented by these stereonet great circles was collected within the dip slope south of Devil Canyon. There are three primary discontinuity sets within the rock mass.

The first discontinuity set consists of bedding which dips to the northeast at about 35 degrees. Stereonets generated at other borings along Devil Canyon suggest that bedding generally dips to the northeast from 20 to 40 degrees and is coincident with the dip of the northwest facing slopes. Bedding is persistent and individual beds can be correlated between borings that were drilled along strike or dip of the beds. The second set may be considered a “lateral” joint that dips 65 degrees towards the northwest. Data from other borings show that the dip direction can vary from northwest to southeast and the dip of this joint set may be as steep as 90 degrees. The lateral joints are thought to be very persistent. The third discontinuity set consists of joints that dip approximately 45 degrees southwest (ranging from 30 to 70 degrees). This joint set is generally normal to bedding and considered the “cross-bed” joint. Observations at outcrops suggest that the cross-bed joints are truncated at the intersection with weaker bedding planes bounding the harder fractured bed.

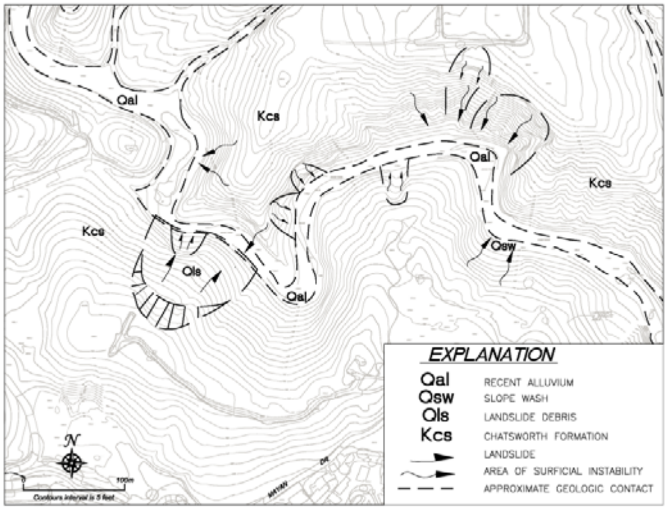

Fig. 2. Geologic Map of Devil Canyon

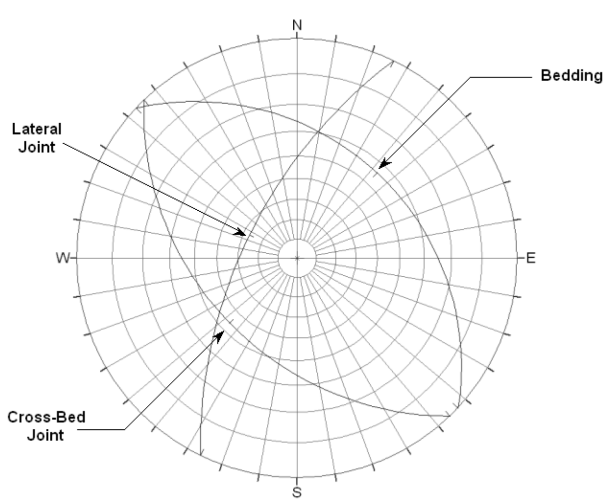

Fig. 3. Stereonet – Devil Canyon Structure

These three discontinuity sets interconnect to form orthogonal blocks. Bedding bounds the top and bottom of the blocks; the two joint sets bound the sides of the blocks. Because of past folding activity in the region, the actual orientations of the discontinuity sets vary slightly along Devil Canyon although the spatial relationship between the discontinuity sets remains generally consistent throughout the project area.

Spacing of the discontinuity sets were estimated using data from the down-hole logging. It was calculated from the orientation of the core hole, dip of the discontinuity, and spacing measured within the borehole [3]. Table 1 shows the distributions and means of the discontinuity spacing within the rock mass.

Table 1. Discontinuity Spacing

Discontinuity	Distribution	Mean Spacing
Bedding	Log Normal	0.4m
Lateral Joint	Log Normal	0.6m
Toe Joint	Neg. Exponential	1.5m

2.2. *Intact Rock Strength*

For the purposes of characterizing the rock mass, the rock was divided into a number of rock grades [4]. Within the ISRM system, rock grades range from R0 through R6, where R0 is extremely weak rock with a uniaxial compressive strength of 0.25 to 1 MPa and R6 having a uniaxial compressive strength greater than 250 MPa. The intact rock strength of the Chatsworth Sandstone ranges from Extremely Weak (R0) to Strong (R4), with the strength grade generally increasing with depth and decreasing oxidation. The majority of the rock encountered within the cores ranged from R0 to R2 and for the purposes of the slopes stability evaluation, it was assumed that the rock grade does not exceed R2.

2.3. *Groundwater*

Surface water was observed to flow intermittently onsite along Devil Canyon. Shallow or perched groundwater occurs onsite as a seasonal condition at the bottom of Devil Canyon. Springs were observed about 6 m above the canyon floor on the north side of the canyon and are associated with phreatophytic vegetation.

Ground water was encountered at depth in several of the core borings. On the dip slope side of Devil Canyon, groundwater is generally located at or slightly above the elevation of the intermittent stream within the canyon bottom. Groundwater increases in elevation to the south. It was noted that in many cases, groundwater coincided with "slightly weathered" or "fresh" bedrock where oxidation of the rock mass is minimal and the color of the rock is gray.

Fig. 4 is the simplified geologic model used for slope stability evaluations on the south side of Devil Canyon.

2.4. Dip Slope Failure Modes

Because residential developments were planned above the crest of the dip slopes, the goal of the slope stability evaluation was to determine the 'set back' distance from which construction could safely proceed.

2.4.1. Translational Failure

The most likely failure mode on the dip slope of Devil Canyon is translational sliding when bedding planes are exposed at the base of the canyon. The large landslide (Qls) shown on Fig. 2 was likely the result of erosional processes that daylighted the bedding.

2.4.2. Bilinear Failure Surface

It is typical practice within Southern California to assume a bilinear failure surface for slope stability calculations of dip slopes [5]. Others have described dip slope failures that include mechanisms besides translational sliding [6]. Modes of failure such as buckling or ploughing were not considered appropriate for this project. This is not because the underlying mechanisms for these failure types are non-existent but because a bilinear failure surface, which develops along bedding and then through the toe, would provide a more conservative setback distance for the required safety factor of 1.5. Bilinear failure mechanism

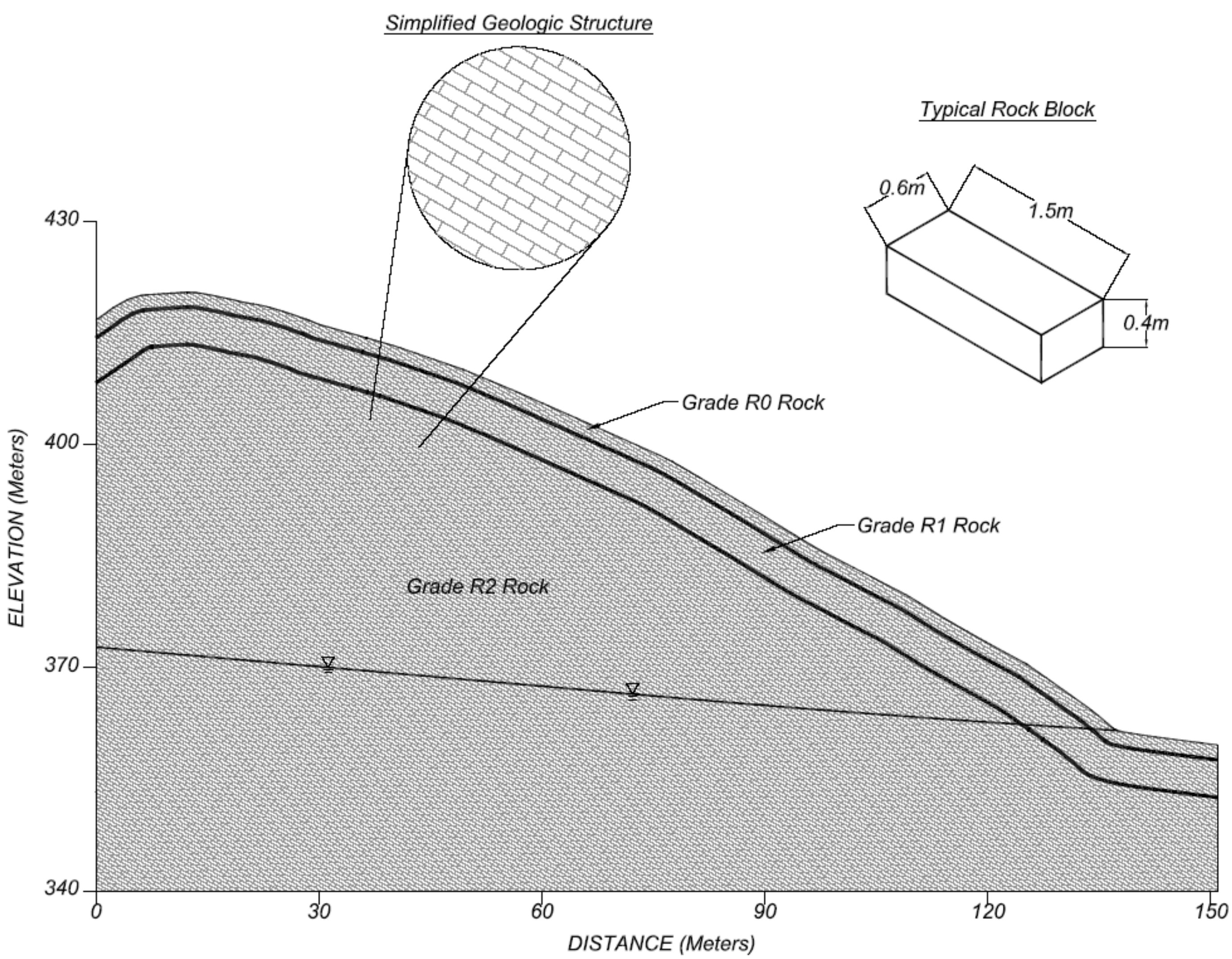

Fig. 4. Simplified Geologic Model used for Stability Evaluation

appears to be an over simplification of a more realistic failure which would involve translational sliding along bedding, a curvilinear failure through the rock mass near the transition between the upper and lower bilinear surface, and finally a step path failure through the toe of the slope.

3. BILINEAR FAILURE MODEL

3.1. Bedding Shear Strength

The shear strength along bedding was estimated from direct residual shear testing of remolded and extremely weak sandstone samples recovered from near surface as per standard practice within Southern California. Torsional ring shear tests were also completed on siltstone bedding layers encountered at depth within the borings. Back analysis of landslide Qls (Fig. 2) was conducted although the actual geologic conditions at the time of sliding are not known with sufficient accuracy to be confident in the back analysis results. Ultimately, the results of torsional ring shear testing were used to represent the shear strength of the bedding because ring shear testing provided the most realistic estimate of the bedding shear strength.

3.2. Shear Strength of the Toe

The shear strength of the rock mass at the toe of the dip slopes may be represented in a number of ways depending on the assumptions made regarding the nature of the failure surface. Within Southern California, one of two potential failure mechanisms is typically considered. These include:

1. Failure at the toe through it's upward movement along a highly persistent sub-vertical joint; and
2. Failure at the toe through shearing of the intact rock material at the toe.

3.3. Prominent "Toe Joint" Shear Strength

The first alternative proposed would consider sliding down along bedding and then up through the toe of the slope along a single highly persistent joint. From the stereonet presented in Fig. 3, it may be observed that this prominent joint would be almost normal to the bedding. With this model only the shear strength of the joint is considered with regards to failure at the toe.

3.4. Intact Rock Shear Strength

The second alternative considers sliding that is enabled through intact rock failure at the toe of the slope. This appears to be the most common failure model considered for deep dip slope failures in Southern California. This is because within Southern California, typical rock units are young, poorly indurated, thickly bedded, and very weak. Observations of landslides within sedimentary lithologies on dip slopes appear to confirm this mode of failure.

With this model, the estimate of the intact rock strength is critical and the influence of joints within the rock mass is neglected.

4. PROPOSED ALTERNATE MODEL

A third model was proposed which would account for the presence of non-persistent jointing combined with intact rock failure at the toe of the dip slopes (i.e. "rock mass" failure). The shear strength for the upper portion of the failure surface coinciding with bedding would be the same as reported above. At the toe of the slope, the rock mass shear strength would be estimated using the Hoek-Brown Failure Criterion [7]. To the authors' knowledge, this is the first project reviewed by Los Angeles County where the Hoek-Brown criterion was proposed to model the shear strength of a rock mass at the base of a dip slope.

Fig. 5 shows a decision tree illustrating the different models considered and the assumptions regarding shear strength that accompanies those models.

5. SHEAR STRENGTH ESTIMATES

5.1. Prominent "Toe Joint" Model

Direct shear testing of saw cut rock samples were performed to estimate the basic friction angle of joints within the rock mass. Although Joint Roughness Coefficients, or JRC [8] values were recorded during the coring activities, practice within Southern California dictates that the lowest measured strengths of the discontinuities be used in the slope stability evaluation. It is up to the consultant to decide on the testing method used. As such, the results of direct shear tests performed on saw cut cores were used to estimate the shear strength of joints at the toe of the slope. Fig. 6 shows the results of this testing.

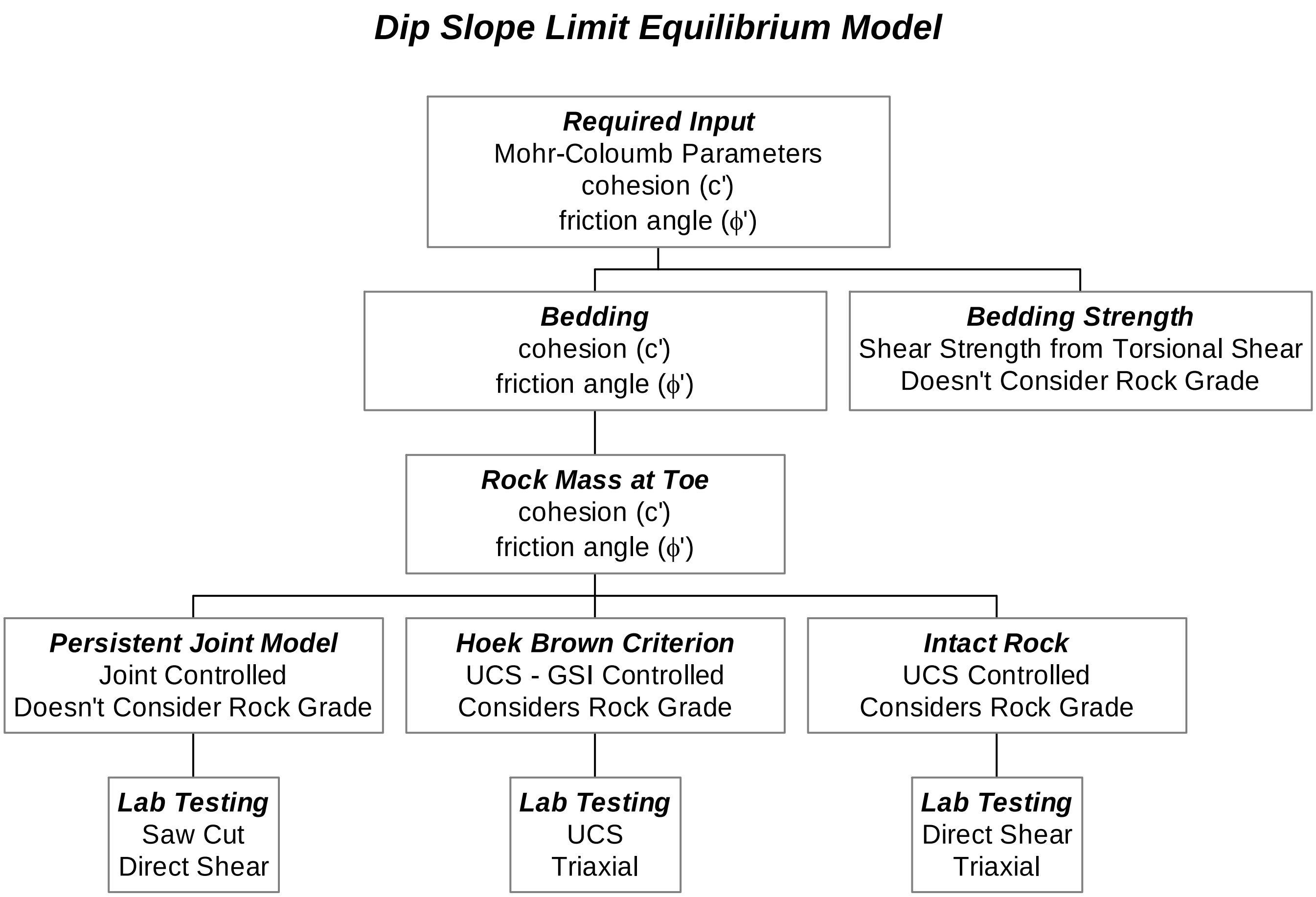

Fig. 5. Decision Tree Followed while Evaluating the Toe Shear Strength Model

This model was thought to be overly conservative as observations at the toe of the dip slopes suggest that the "toe" joints were significantly limited in persistence, truncating where they intersected bedding. Moreover, using the saw cut samples to represent the shear strength of the joints discounts dilation of the joints and the increase in frictional strength created by the natural surface roughness/waviness of the joints.

5.2. Intact Rock Model

Shear strength values for rock samples of R0 and R1 grade were determined through direct shear testing. Typically, the "ultimate" shear strength recorded during these tests was used. The ultimate shear strength is that strength measured after the peak strength and prior to the residual strength, and requires judgment and experience to properly ascertain from the stress-strain curves. The ultimate shear strengths measured during direct shear testing of the grade R0 and R1 samples are presented in Fig 7.

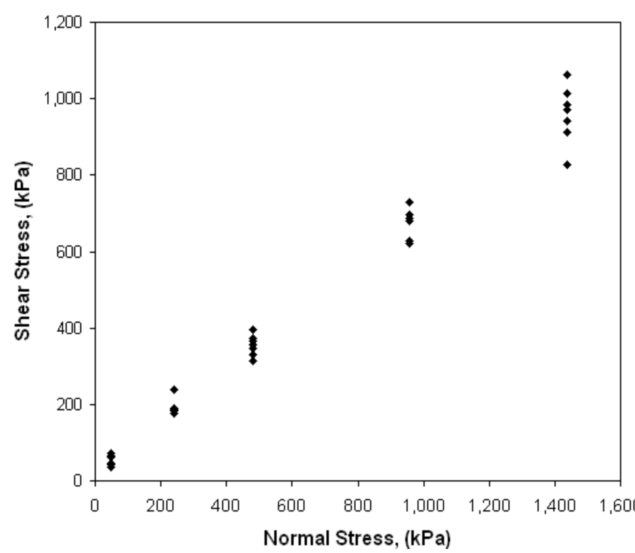

Fig. 6. Rock Core Saw Cut Direct Shear Strength

The use of intact rock strengths was deemed overly optimistic because the influence of jointing within the rock mass would act to decrease the overall strength of the rock at the toe of the dip slopes. Although this may not be too gross of an oversimplification for the R0 and R1 grade rocks, it would be for the R2 grade rock.

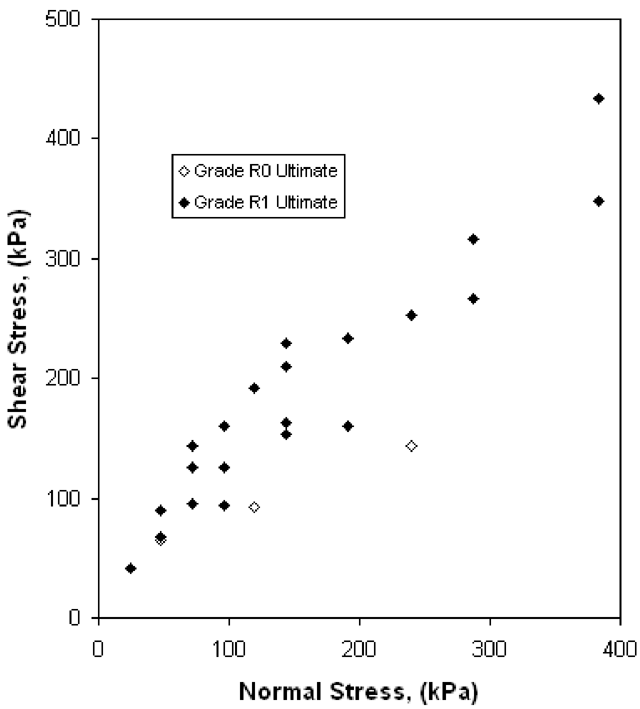

Fig. 7. “Ultimate” Shear Strength of R0 and R1 Rock.

5.3. *Hoek-Brown Rock Mass Model*

As an alternative to the models above, the Hoek-Brown Strength Criterion was proposed as a more equitable means to estimating the shear strength of the rock mass at the toe of the slope; i.e. one that recognized the combined role that both non-persistent jointing and intact rock strength play in forming the overall rockmass strength. The reviewing agency was initially reluctant, but relented and insisted that only the most conservative input parameters be used in developing any empirical-based shear strength envelope. Moreover, the reviewing agency was more comfortable with the Mohr-Coulomb Failure Criterion than the Hoek-Brown Criterion, and insisted that any shear strength parameters determined using Hoek-Brown be converted to Mohr-Coulomb failure envelopes.

6. HOEK-BROWN FAILURE CRITERION

The Hoek-Brown Failure Criterion appeared to be a reasonable approach to represent the shear strength of the rock mass at the toe of the slope. The geology at the base of Devil Canyon is conducive to this type of strength estimation for the following reasons:

- Shearing at the toe would likely have to take place through both intact rock and joints (e.g. step-paths),
- Joints are truncated at bedding,
- The individual blocks within the rock mass are much smaller than the slope, and
- The actual shearing method at the toe would be difficult to evaluate without assuming an equivalent continuum model.

Therefore, in the absence of a more accurate methodology to analyze step-path failure of sliding along joints and through intact rock bridges in the toe of the dip slopes, the Hoek Brown Failure Criterion provides a practical solution that appears to fit the geologic conditions at the base of Devil Canyon.

6.1. *Triaxial Testing of Intact Rock Samples*

An estimation of the intact rock strength is required to generate a rock mass failure envelope [9]. Single staged, undrained triaxial tests were performed using confining pressures between zero and 50 percent of the intact rock uniaxial compressive strength (UCS) on samples of R2 grade. The average UCS for the R2 grade rock was determined to be 14 MPa. By definition, the lowest UCS would be 5 MPa. To be consistent with the state of practice, a UCS of 5 MPa was used for the estimation of the Hoek-Brown Strength.

A total of 15 triaxial tests were conducted on grade R2 intact rock samples from the Chatsworth Formation. It was agreed that shearing through the R0 and R1 grade rock would not be influenced by jointing and therefore, these rock types were not chosen for the triaxial testing. One of the triaxial tests performed involved “staged” loading whereby three different confining pressures were used during the testing procedure on the same sample. This staged test was completed simply to demonstrate the strength of the intact rock for comparison to the scaled empirical rock strength that would be estimated using the Hoek-Brown method.

Triaxial testing on the R2 grade rock indicated that strengths were much higher than typically used for slope stability evaluations in Southern California.

Comparisons to a compiled inventory of laboratory testing results from geotechnical consultants in the Northridge area of Southern California [10], showed that the values reported are suitable for correlation with those obtained from direct shear testing of near surface R0 grade samples but not representative of the Chatsworth Formation at depth.

The goal of the triaxial testing was to establish a relationship between peak axial strength and confining pressure, from which a site-specific m_i value could be derived to estimate the Hoek-Brown criterion [9]. This required careful deliberation of the data, consultation with Dr. Hoek, and a rational method to estimate sample disturbance generated during drilling and transport of the samples to the lab testing facility. This treatment of sample disturbance is outside the scope of the current paper and is the subject of a subsequent paper that is currently in preparation.

6.2. GSI Estimate

The condition of the rock mass dictates the rock mass strength. This requires that lab-based triaxial strength data be “scaled” to account for the difference between intact rock and rock mass strength. Scaling may be achieved using the Geologic Strength Index (GSI) [9], which was determined through visual assessment at outcrops near the base of Devil Canyon and also by correlation of Rock Mass Rating (RMR) [11] data collected from the rock coring activities. GSI can be taken as $RMR_{89} - 5$, where the rating for ground water is set to 15, and the adjustment for discontinuity orientation is set to zero. This relationship is considered valid where GSI is greater than 30.

The program @RISK Version 4.5.4 by Palisade Decision Tools was utilized to estimate a statistical distribution for GSI.

6.3. Mohr-Coulomb Strength Generation

The geotechnical literature provides a precise methodology to establish the Mohr-Coulomb estimate of shear strength from the Hoek Brown Criterion [12]. The methodology is based on a given slope height and the unit weight of the rock mass.

This methodology was confirmed as being valid by the authors when completing individual slope stability evaluations for different cross-sections generated as part of this study. The reviewing agency was not familiar with the Hoek-Brown Criterion. Therefore, it was decided that one set of Mohr-Coulomb strength values would be reported that would be representative, conservative, and based on the range of slope heights evaluated on the south side of Devil Canyon. To achieve this estimate, the Hoek-Brown envelope generated [12] was plotted using a computer spreadsheet. Curve fitting was completed using linear regression over the range of normal stresses anticipated at the toe of the slope. The results were then compared to back analyses of failed rock slopes within the geotechnical literature [13]. The results and methodology were presented to the reviewing agency along with an evaluation showing that in general, the shear strength estimated using the Hoek Brown Criterion was similar to a composite shear strength that would be achieved by assuming that shearing took place along a surface consisting of about 90 percent discontinuous rock and 10 percent intact rock.

Table 2 presents the results of the comparison of the different Mohr-Coulomb shear strength values obtained for each of the different toe failure models, as previously presented and discussed, and their implication on the recommended setback distance. The strength values reported are for the grade R2 rock.

Table 2. Mohr-Coulomb Strengths for R2 grade rock and Relative Setbacks

Model	c’, kPa	ϕ’, degrees	Setback
Prominent joint	0	32	Conservative
Intact Rock	2,700	43	Liberal
H-B Criterion	100	41	Intermediate

7. DISCUSSION & CONCLUSIONS

A residential development has been proposed above a series of dip-slopes above the Devil Canyon, north of the City of Chatsworth, California. The evaluation was completed while assuming a bilinear slope failure mode, because this is the recommended practice in Southern California. This bilinear shear mode includes shearing along bedding and failure through the toe of the slope. One of two models is recommended for treating the issue of toe failure: upward shearing of the toe along a prominent/persistent joint, and shearing through intact rock at the toe.

The first of these models was preferred by the reviewing agency because it provided the most conservative structural setbacks for the project. In turn, the developer did not favor it as it minimized the amount of developable land and thus would be the most costly. Based on geological mapping and borehole drilling at the Devil Canyon site, it was determined that the prominent toe joint model was overly conservative. Such a model would be appropriate where both the bedding and cross-cutting joints are highly persistent, but observations indicated that the cross-cutting joints in the dip slope at Devil Canyon were much more limited in persistence, and truncated at their intersections with the bedding planes.

At the opposite end of the spectrum, the second of these models (intact rock strength model) produced the smallest setbacks. The Developer preferred this model as it provided the largest profit margin. The review agency was initially accepting of this model, as it followed standard practice. This model held the greatest potential for future liability. Although the intact rock strength model may be appropriate where the rock mass is bedded and jointing is negligible or the rock mass is extremely weak, young and poorly indurated, the observed rock mass conditions proved otherwise.

Instead, the Hoek-Brown Model was considered the most appropriate shear strength model for this project becuase it best represented the geologic structure encountered at the toe of the dip slopes of Devil Canyon; i.e. one where toe failure would involve rock mass failure, where the shear surface would pass along joints of limited persistence and through intact rock bridges.

It should be noted that these project conditions are somewhat unique and do not conform to established code of practice within Southern California. This is because the Chatsworth Formation is among the oldest, most indurated, and well jointed in the region. Although it would have been within the state of practice to consider either the "Prominent Joint" or "Intact Rock" models in calculating the setback distances, the Hoek-Brown Criterion was judged more appropriate. This speaks to the importance of choosing a model based on the geology of the area and not assuming that standardized models necessarily apply to all cases.

8. ACKNOWLEDGEMENTS

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