On the Weak Limestone Rock Slope Stability Analysis

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ABSTRACT: The natural rock slope of Weak Limestone has been studied, modelled analysis and reviewed. This paper describes the Rock Slope Stability Analysis for the construction of a Generating Station. Four scenarios were analysed i.e.: original proposal of 75o; continuous slope of 60°; and double step slope of 60° and 60° with a bench; double Steep slope of 65° and 75° with a bench. Slope reinforcements (End Anchored and High Tensile Mesh) were recommended. The analysis is with Limit equilibrium method, LEM and Finite element method, FEM.

Keywords: Limestone, Rock Slope Stability, LEM, FEM.

1. INTRODUCTION

This paper describes the Slope Stability Analysis for the construction of a Generating Station. This paper aims to analyse the rock slope stability: A steep slope early proposed 75o, and then propose the following;

a. Steep slope of 60°
b. Double Steep slope of 60° and 60° with a bench
c. Double Steep slope of 65° and 75° with a bench
d. Recommendation of slope reinforcement (End Anchored and High Tensile Mesh) if needed.

The main analysis conducted is aimed at finding the Factor of Safety (FS), through the Limit Equilibrium method using Rocscience's slope stability Analysis Software Slide 3D and with Finite Element Method (Griffiths 2015), and a Microsoft Excel Software.

2. CASE STUDY

2.1 Location

The drilling hole point at the nearest future Slope is at BHX depicted in Figure 1. Figure 2 showed the Borelog BH-X

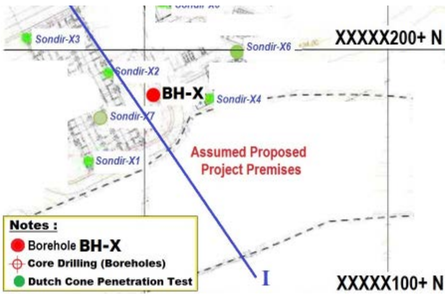


Figure 1 Assumed Geotechnical Investigation Area

2.2 Seismic Condition

The Earthquake Strong Ground Motion properties was calculated using the expected peak ground acceleration for Probability Exceeding 2% in 50 Years, according to the Indonesian Earthquake Hazard Map (Ministry of Public Work 2017) for the maximum credible earthquake at our study site in Eastern Indonesia. The resulting PGA from the Earthquake Hazard Map was 0.5 g. A moment magnitude of Mw = 8.0 was used based on W.G. Housner's chart (1971). Figure 4 and Figure 5 depict the Indonesian Earthquake Hazard Map and Housner's Recommendation Table for Strong Ground Motion.

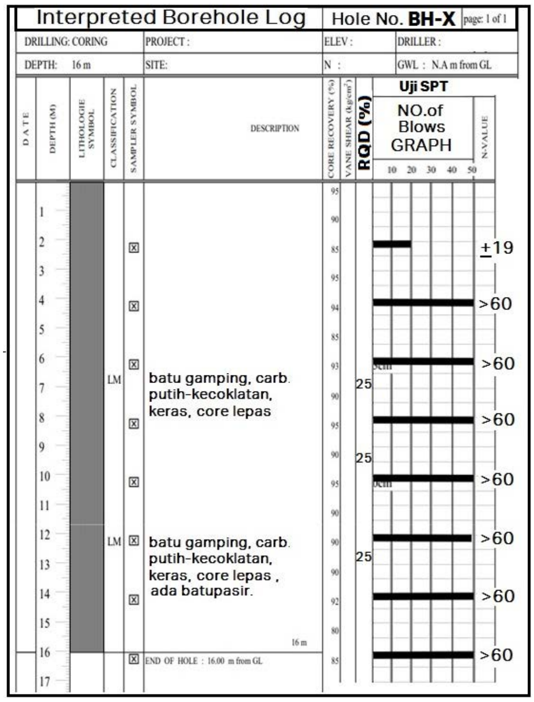


Figure 2 An example only for a Borehole BH-X, with lithology of LIMESTONE that shown the Low Values of RQD (ranging from 0 to 25%)

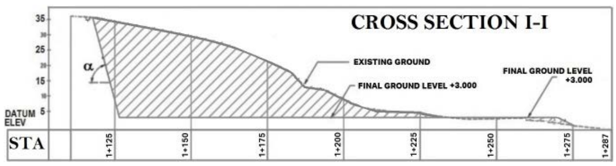


Figure 3 Site (Previously) Planned Cross Section Profile

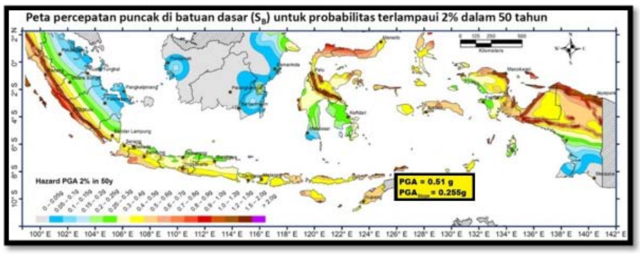
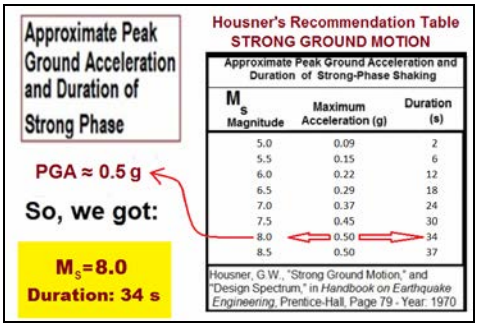


Figure 4: The Peak Ground Acceleration Map in Bedrock (Sb) for Probability Exceeding 2% in 50 Years (For our site, The PGA = 0.51 g, and the Acceleration Multiplier is 0.5).



Ms Magnitude	Maximum Acceleration (g)	Duration (s)
5.0	0.09	2
5.5	0.15	6
6.0	0.22	12
6.5	0.29	18
7.0	0.37	24
7.5	0.45	30
8.0	0.50	34
8.5	0.50	37

Figure 5: GW Housner's Table and Calculation for Mw and Earthquake Duration - Maximum Peak Ground Acceleration at Bedrock

2.3 Laboratory Test Results

Based on the Rock Laboratory and Point Load tests, we found out that the Limestone in the area is weak Limestone, with UCS = 1800 kPa, Young's Modulus, E = 250000 kPa and Poisson's Ratio, v = 0.23.

Through additional analysis of Rockmass Properties we found out that the Rock mass is FAIR (Q or NGI Classification), with GSI = 41. (See Appendix C).

2.4 Ground (Soil) Resistivity Test Results

Based on the Soil Resistivity test, we found out that the Limestone in the area is a weak Limestone, Figure 6 showed the summary of it.

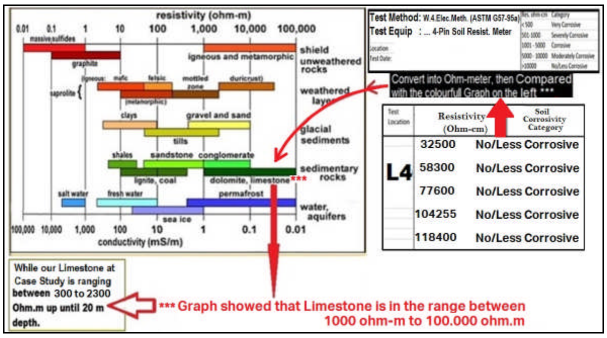


Figure 6. Summary of Soil (Ground) Resistivity Results (Showed the type of Limestone that is in the range of weak rock, which is anomaly from the common Limestone found elsewhere.)

3. METHODS OF ANALYSIS

3.1 Limit Equilibrium Method (LEM)

LEM is a method that uses the principle of force equilibrium. This method of analysis first assumes the field of sliding that can occur, the field of assumption that is assumed to be circular and non-circular (Figure 7)

LEM is the most popular approach in slope stability analysis. This method is well known to be a statically indeterminate problem, and assumptions on the interslice shear forces are required to render the problem statically determinate. Based on the assumptions of the internal forces and force and/or moment equilibrium, there are more than 10 methods developed for slope stability analysis. The famous methods include those by Janbu (1957, 1973), Spencer (1967) and Morgenstern and Price (1965).

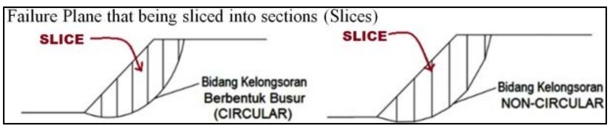


Figure 7. Failure Line: Circular & Non-Circular+
+The calculation is done by dividing layers of rock / soil that are in the field of landslides into slices.

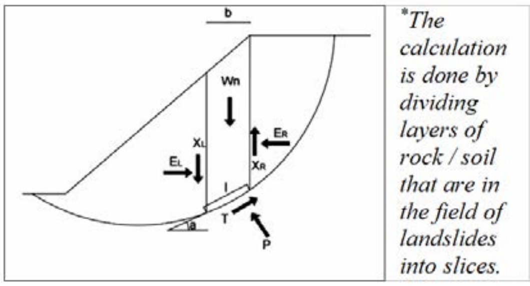


Figure 8. Forces that work in the Slices*

All these methods consider the moment and force equilibrium in each slice. If the moment and force equilibrium is satisfied in each slice, the overall moment and force equilibrium will be satisfied automatically. The basic concept in these methods is the same; the difference lies in the assumption of the interslice forces. If both moment and force equilibrium are satisfied, the assumption on interslice forces should have only small effect on the factor of safety obtained.

In the Morgenstern and Price method, an assumption is made regarding the relationship between interslice shear and normal forces. After obtaining the computer output based on this assumption, all the computed quantities, including the inter-slices forces, must be examined to determine whether they seem reasonable. If not, a new assumption must be made.

All these methods can be applied to both circular and noncircular failure surfaces.

3.2 The failure criterion for intact rock (Limestone)

In rock mechanics practice the use of Hoek-Brown failure criteria is common. The Hoek-Brown properties (GSI, mi and s) can be converted into Mohr-Coulomb intact rock properties (ϕ and c).

a. Mohr Coulomb Failure Criterion

The failure criterion for intact rock used is the Mohr-Coulomb criterion as follows, $\tau_i = \sigma_{ni} \tan \phi_o + c_o$; where σ_{ni} is the normal stress on the failure plane and ϕ_o and C_o are material constants for intact rock (Appendix C).

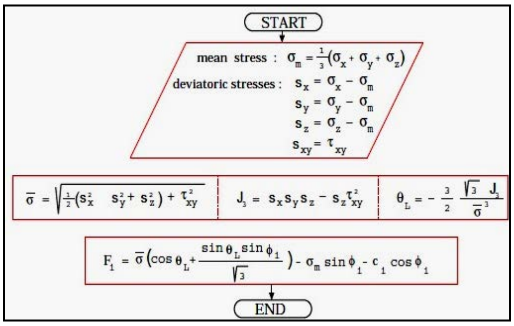


Figure 9. Algorithm for Failure (Fi) Calculation of the Intact Rock (Limestone).

b. Generalized Hoek-Brown Failure Criterion (constitutive Model)

For a constitutive Model of the Generalized Hoek-Brown (Hoek 1995; Eberhardt 2012), the equation is as follows:

$$\sigma_1 - \sigma_3 - \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a = 0 \qquad (1)$$

Where σ_1 dan σ_3 are major and minor principal stress, σ_{ci} is unconfined compressive strength (UCS) or the maximum axial compressive stress that a right-cylindrical sample of material can withstand under unconfined conditions — the confining stress is zero, and

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{2}$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{3}$$

$$a = \frac{1}{2} + \frac{1}{6}\left[\exp\left(\frac{GSI}{15}\right) - \exp\left(\frac{-20}{3}\right)\right] \tag{4}$$

mi is a material constant for the intact rock, GSI (the Geological Strength Index) relates the failure criterion to geological observations in the field, and D is a "disturbance factor" which depends upon theo of disturbance to which the rock mass has been subjected by blast damage and/or stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses.

3.3 The Finite Element Method (FEM)

The FEM is a numerical method for solving problems of engineering and mathematical physics. Typical problem areas of interest in engineering and mathematical physics that are solvable by use of the finite element method include structural analysis, geotechnical engineering (inc'l Slope Stability, Tunneling etc.), heat transfer, fluid flow, mass transport, and electromagnetic potential.

The majority of slope stability analyses performed in practice still use traditional limit equilibrium approaches involving methods of slices that have remained essentially unchanged for decades. Then, in 1967, Whitman & Bailey (1967) set criteria for the then emerging methods to become readily accessible to all engineers (Louhenapessy 1995). The FEM represents a powerful alternative approach for slope stability analysis which is accurate,versatile and requires fewer a priori assumptions, especially, regarding the failure mechanism. The author will use Griffiths' FEM Software, name: SLOPE64 (Griffiths 2004, Griffiths & Lane 1999) to check the Limestone Rock Slope Stability.

4. ASSUMPTIONS

The calculation of the factor of safety of the given slope was conducted, which is done depending on the rock material properties (based on Mohr-Coulomb and/or Hoek-Brown Constitutive Model), the slope geometry, static (live) load (10 kPa on top of the slope), Seismic Load (0.255 g) and if necessary the slope reinforcement.

The two loadings that were used were Non-Earthquake (NE) loading, and Earthquake (E) loading. Earthquake loading was provided from Figure 4, The Peak Ground Acceleration Map in Bedrock (Sb) for Probability Exceeding 2% in 50 Years (Peta Percepatan Puncak di Batuan Dasar (Sb) Untuk Probabilitas Terlampaui 2% Dalam 50 Tahun) by The Public Work Department Republic of Indonesia.

Recommended values of safety factors for rock slopes use, SF > 1.5 reference RSNI "Persyaratan Perancangan Geoteknik BSNI 2017. Recommendation of earthquake values of safety factors using, SF > 1.0 Hynes-Griffin and Franklin (1984). (Figure 10).

Recommended values of safety factors for rock slopes

Condition of rock slope	Recommended safety factor values
Permanent condition	1.5
Temporary condition	1.3

Reference : RSNI3 Persyaratan perancangan geoteknik - BSNI - Page134

Suggested Methods for Performing Pseudostatic Screening Analyses

(1) Reference	(2) Reference Acceleration, a_{ref}	(5) Minimum Factor of Safety
Seed (1979)	$0.75g\left(M \approx 6\frac{1}{2}\right)$	1.15
Seed (1979)	$0.75g\left(M \approx 8\frac{1}{4}\right)$	1.15
Hynes-Griffin and Franklin (1984)	$PGA_{rock}(M \leq 8.3)$	1.0
Bray et al. (1998)	PGA_{rock}	1.0

Figure 10: Safety Requirement: Above Left Column, shows that earthquake loading Factor of Safety requirement is 1.5 (Permanent Condition), whilst other common practice, such as Hynes-Griffin and Franklin (1984) showed that the Safety Factor for earthquake loading should be no less than 1.0. See (Right Column Above).

5. RECOMMENDATION AND CONSLUSION

It is recommended that using The Slope Angle of 60°s without Shotcrete (but instead will use High Tensile Mesh) and using end-anchor of 5 m x 5 m, with 15 m long and 100 kN tensile capacity.

Table 1 and Table 2 below details the results of the slope stability analysis (72 Run) along with the loadings (static and seismic) and some of the reinforcements that were provided.

Table 3 showed that from the Matrix of Percentage for Factor of Safety compared with Required Factor of Safety, the best option for design is using the double slope of 60o dip angle. While a Table in Appendix D, is the FEM Summary Results.

The above recommendation is still a bit higher than the one recommended by Rodrigues (Appendix B), where in his Table 6.5, Rodrigues et.al. recommend that (for broken Limestone) a double slope with first slope angle is 53° and a higher level slope is 45o angle. The chosen of broken Limestone is due to the Rock Mechanics Lab. Results and the broken/low values of RQD (Figure 2).

It is recommended to check the tension crack and make sure if there are joint rock appears in the weak limestone the dip angle of the joint rock should be lower than 20° (Appendix E).

Table 1 Summary of Without Shotcrete (Instead, now with: High-Tensile steel wire and Erosion Control)

Slope Geometry	Top Slope Angle (Degree)	Bottom Slope Angle(Degree)	Material Model / Method	Reinforcement (End Anchor) Y/N	Groundwater	Load (Static is 10 kPa), and Eartquake 50% of 0.51 PGA	Safety Factor: Morgensten-Price	Safety Factor: MEAN from two (Least) Limit Equilibrium Method	FIGURE A ... Indonesian Code for Geotech Requirement: (RSNI 2017 & Hynes-Griffin 1984, See Appendix B)
Double Slope	75	64	Mohr-Coulomb	NO	YES	Static	2.029	2.032	1.500 A10
	75	64	Mohr-Coulomb	NO	YES	Earthquake	1.432	1.411	1.000
	75	64	Hoek Brown-D0	NO	YES	Static	1.388	1.356	1.500 A9
	75	64	Hoek Brown-D0	NO	YES	Earthquake	0.999	0.942	1.000
Double Slope	75	64	Mohr-Coulomb	YES 5x5, 100 kN, 15m	YES	Static	2.042	2.043	1.500 A7
	75	64	Mohr-Coulomb	YES (as above)	YES	Earthquake	1.435	1.414	1.000
	75	64	Hoek Brown-D0	YES (as above)	YES	Static	1.404	1.374	1.500 A8
	75	64	Hoek Brown-D0	YES (as above)	YES	Earthquake	1.018	0.959	1.000
Double Slope	60	60	Mohr-Coulomb	NO	YES	Static	2.026	2.043	1.500 A6
	60	60	Mohr-Coulomb	NO	YES	Earthquake	1.433	1.414	1.000
	60	60	Hoek Brown-D0	NO	YES	Static	1.400	1.365	1.500 A5
	60	60	Hoek Brown-D0	NO	YES	Earthquake	1.011	0.948	1.000
Single Slope	75		Mohr-Coulomb	NO	YES	Static	1.921	1.699	1.500 A4
	75		Mohr-Coulomb	NO	YES	Earthquake	1.422	1.219	1.000
	75		Hoek Brown-D0	NO	YES	Static	0.991	0.976	1.500 A11
	75		Hoek Brown-D0	NO	YES	Earthquake	0.702	0.673	1.000
Single Slope	60		Mohr-Coulomb	NO	YES	Static	1.897	1.898	1.500 A2
	60		Mohr-Coulomb	NO	YES	Earthquake	1.397	1.376	1.000
	60		Hoek Brown-D0	NO	YES	Static	1.286	1.253	1.500 A3
	60		Hoek Brown-D0	NO	YES	Earthquake	0.927	0.880	1.000
Single Slope	60		Mohr-Coulomb	YES 5x5, 100 kN, 15m	YES	Static	1.925	1.924	1.500 A13
	60		Mohr-Coulomb	YES (as above)	YES	Earthquake	1.414	1.391	1.000
	60		Hoek Brown-D0	YES (as above)	YES	Static	1.317	1.287	1.500 A14
	60		Hoek Brown-D0	YES (as above)	YES	Earthquake	0.953	0.904	1.000

Table 2 Summary (with Shotcrete: Not Recommended, due to sShotcrete is Prone to Groundwater and Can Crack)

Slope Geometry	Top Slope Angle (Degree)	Bottom Slope Angle(Degree)	Material Model / Method	Reinforcement (End Anchor) Y/N	Groundwater	Load (Static is 10 kPa), and Eartquake 50% of 0.51 PGA	Safety Factor: Morgensten-Price	Safety Factor: MEAN from two (Least) Limit Equilibrium Method	A: Indonesian Code for Geotech Requirement (RSNI 2017 & Hynes-Griffin 1984, See Appendix B)
Double Slope	64	75	Mohr-Coulomb	NO	YES	Static	2.015	**2.060**	1.500
	64	75	Mohr-Coulomb	NO	YES	Earthquake	1.444	**1.430**	1.000
	64	75	Hoek Brown-D0	NO	YES	Static	1.104	**1.101**	1.500
	64	75	Hoek Brown-D0	NO	YES	Earthquake	0.741	**0.716**	1.000
Double Slope	64	75	Mohr-Coulomb	YES 5x5, 100 kN, 15m	YES	Static	2.028	**2.074**	1.500
	64	75	Mohr-Coulomb	YES (as above)	YES	Earthquake	1.454	**1.437**	1.000
	64	75	Hoek Brown-D0	YES (as above)	YES	Static	1.204	**1.203**	1.500
	64	75	Hoek Brown-D0	YES (as above)	YES	Earthquake	0.837	**0.818**	1.000
Double Slope	60	60	Mohr-Coulomb	NO	YES	Static	2.012	**2.057**	1.500
	60	60	Mohr-Coulomb	NO	YES	Earthquake	1.435	**1.418**	1.000
	60	60	Hoek Brown-D0	NO	YES	Static	1.248	**1.239**	1.500
	60	60	Hoek Brown-D0	NO	YES	Earthquake	0.853	**0.829**	1.000
Double Slope	60	60	Mohr-Coulomb	YES 5x5, 100 kN, 15m	YES	Static	2.026	**2.068**	1.500
	60	60	Mohr-Coulomb	YES (as above)	YES	Earthquake	1.438	**1.420**	1.000
	60	60	Hoek Brown-D0	YES (as above)	YES	Static	1.321	**1.319**	1.500
	60	60	Hoek Brown-D0	YES (as above)	YES	Earthquake	0.930	**0.907**	1.000
Single Slope	75		Mohr-Coulomb	NO	YES	Static	1.823	**1.748**	1.500
	75		Mohr-Coulomb	NO	YES	Earthquake	1.187	**1.190**	1.000
	75		Hoek Brown-D0	NO	YES	Static	0.863	**0.861**	1.500
	75		Hoek Brown-D0	NO	YES	Earthquake	0.564	**0.529**	1.000
Single Slope	75		Mohr-Coulomb	YES 5x5, 100 kN, 15m	YES	Static	2.021	**1.860**	1.500
	75		Mohr-Coulomb	YES (as above)	YES	Earthquake	1.206	**1.208**	1.000
	75		Hoek Brown-D0	YES (as above)	YES	Static	0.889	**0.898**	1.500
	75		Hoek Brown-D0	YES (as above)	YES	Earthquake	0.609	**0.596**	1.000
Single Slope	60		Mohr-Coulomb	NO	YES	Static	1.831	**1.866**	1.500
	60		Mohr-Coulomb	NO	YES	Earthquake	1.347	**1.334**	1.000
	60		Hoek Brown-D0	NO	YES	Static	1.066	**1.059**	1.500
	60		Hoek Brown-D0	NO	YES	Earthquake	0.740	**0.710**	1.000
Single Slope	60		Mohr-Coulomb	YES 5x5, 100 kN, 15m	YES	Static	1.835	**1.882**	1.500
	60		Mohr-Coulomb	YES (as above)	YES	Earthquake	1.349	**1.341**	1.000
	60		Hoek Brown-D0	YES (as above)	YES	Static	1.113	**1.109**	1.500
	60		Hoek Brown-D0	YES (as above)	YES	Earthquake	0.781	**0.755**	1.000

Notes: = Limit Equlibrium Method, FEM = Finite Element Method.

Table 3 Matrix of Percentage for FS compared with Required FS
PS: The tables bellow each Slope Profile is taken from the Table 1

6. REFERENCES

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APPENDIX A – Portion of LEM

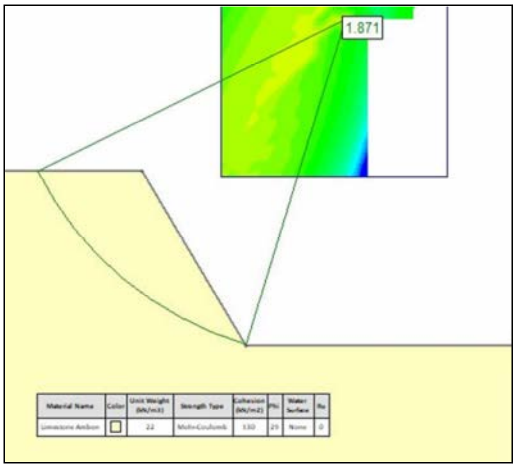
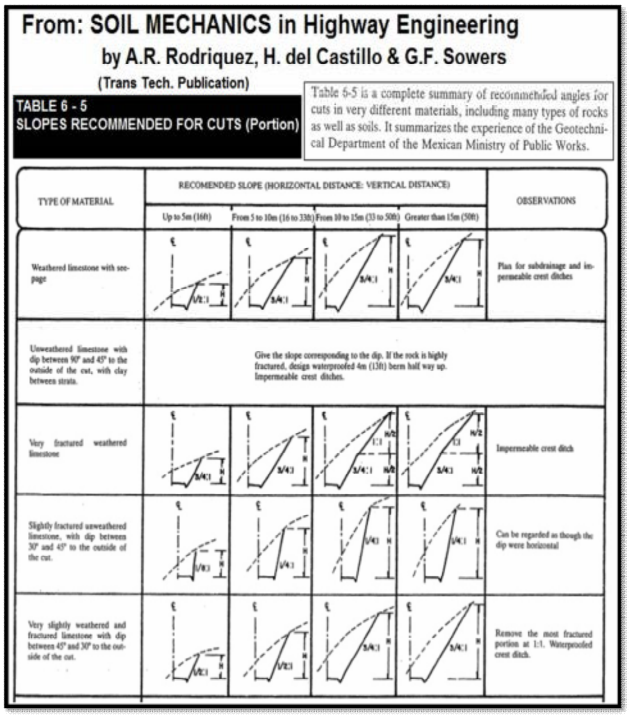


Figure A.1 Portion of LEM Results (by Slide Software)

APPENDIX B – Slopes Recommended for Cuts



From: SOIL MECHANICS in Highway Engineering

by A.R. Rodriquez, H. del Castillo & G.F. Sowers

(Trans Tech. Publication)

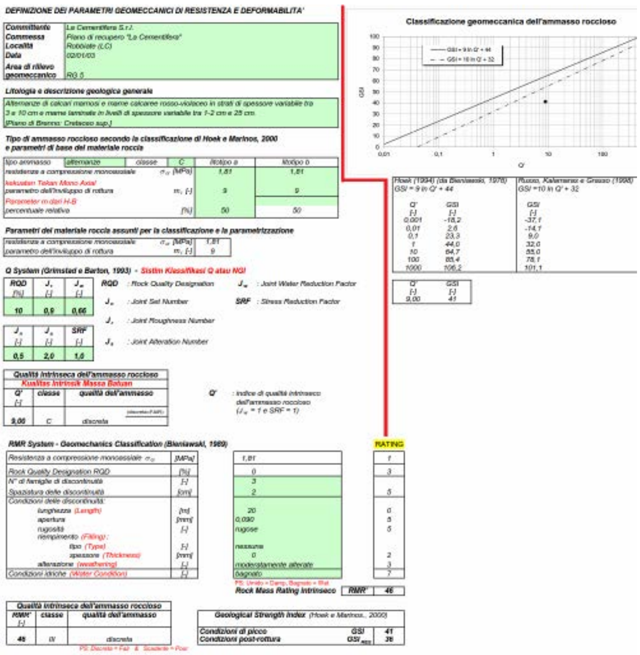
TABLE 6 - 5
SLOPES RECOMMENDED FOR CUTS (Portion)

Table 6-5 is a complete summary of recommended angles for cuts in very different materials, including many types of rocks as well as soils. It summarizes the experience of the Geotechnical Department of the Mexican Ministry of Public Works.

TYPE OF MATERIAL	RECOMENDED SLOPE (HORIZONTAL DISTANCE: VERTICAL DISTANCE)				OBSERVATIONS
	Up to 5m (16ft)	From 5 to 10m (16 to 33ft)	From 10 to 15m (33 to 50ft)	Greater than 15m (50ft)	
Weathered limestone with seepage	1/2:1	3/4:1	3/4:1	3/4:1	Plan for subdrainage and impermeable crest ditches
Unweathered limestone with dip between 90° and 45° to the outside of the cut, with clay between strata.	Give the slope corresponding to the dip. If the rock is highly fractured, design waterproofed 4m (13ft) berm half way up. Impermeable crest ditches.				
Very fractured weathered limestone	3/4:1	3/4:1	3/4:1, 1:1	3/4:1, 1:1	Impermeable crest ditch
Slightly fractured unweathered limestone, with dip between 30° and 45° to the outside of the cut.	1/8:1	1/4:1	1/4:1	1/4:1	Can be regarded as though the dip were horizontal
Very slightly weathered and fractured limestone with dip between 45° and 30° to the outside of the cut.	1/2:1	1/2:1	3/4:1	3/4:1	Remove the most fractured portion at 1:1. Waterproofed crest ditch.

Rodrigues et.al. recommend that (for broken Limestone) a double slope with first slope angle is 53^{o} and a higher level slope is 45^{o} angle.

APPENDIX C – Rock Classification with Mohr Coulomb and Hoek-Brown Intact Rock Properties



Parametri di resistenza secondo il criterio di rottura di Hoek e Brown

$\sigma_1 = \sigma_3 + \sigma_{ci} (m_b \cdot \sigma_3/\sigma_{ci} + s)^a$ (Hoek et al., 2002)

Materiale roccia		
resistenza a compressione monoassiale	σ_{ci} [MPa]	1,81
resistenza a trazione (calcolata)	σ_{ti} [MPa]	0,20
parametro dell'inviluppo di rottura	m_i [-]	9

Coefficiente di disturbo	D [-]	0,0

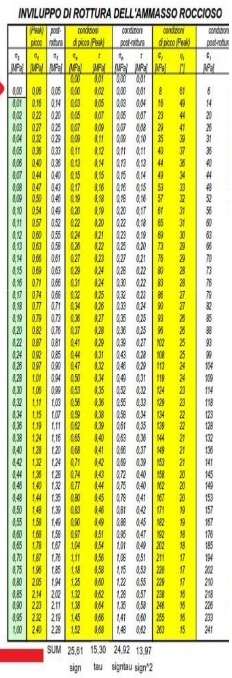
Ammasso roccioso - Condizioni di picco		
parametri dell'inviluppo di rottura	m_b [-]	1,09
	s [-]	1,4E-03
	a [-]	0,51
resistenza a trazione	σ_t [MPa]	0,00
resistenza a compressione monoassiale	σ_c [MPa]	0,06
resistenza globale	σ_{cm} [MPa]	0,25

Ammasso roccioso - Condizioni post-rottura		
parametri dell'inviluppo di rottura	m_b [-]	0,92
	s [-]	8,2E-04
	a [-]	0,51
resistenza a trazione	σ_t [MPa]	0,00
resistenza a compressione monoassiale	σ_c [MPa]	0,05
resistenza globale	σ_{cm} [MPa]	0,22

Modulo di deformabilità, E (Young's Modulus)

(Serafim e Pereira, 1983; Hoek et al., 2002) E from ITB Lab

Condizioni di picco (Peak Condition / Kondisi Puncak)	E_m [GPa]	0,801	0,256
Condizioni post-rottura	E_m [GPa]	0,601	



ANGLE of INTERNAL FRICTION:

Phi = Atan [(24,92 - 391,893564) / 51 / (13,97 - 655,739) / 51] * 180/3.14156

= Atan (-7,98 / (-13,95)) * 180 / 3.14156

= Atan (0,571806) * 180/3.14156)

= Atan (0,57) * 180 / 3.14156

= 29,76 degree (Ini adalah harga disainnya)

Cohesion

Coh = (15,30 / 51 - 25,61 / 51) * Tan (Phi x 3.14159/180)

= (0,33 - 0,56) 0,57

= (-0,224) * 0,57

= -0,13 Mpa

APPENDIX D –Finite Element Method: SLOPE64

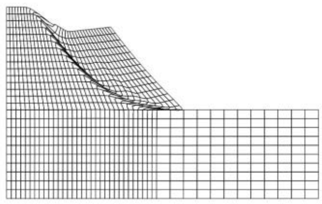
FILE: A60.res (Slope Angle 60 degree)

```
SLOPE64

written by
D.V. Griffiths
Colorado School of Mines
2013
------------------------------------------------------------
Example 2: AMBON 60 deg Slope
w1=   26.00
s1=   17.54
w2=   47.00
h1=   29.79
h2=   25.70

nx1=   32
nx2=   12
ny1=   15
ny2=   10

Group  phi      c      psi    gamma       e          v
  1   26.00  130.00   0.00   22.00  0.2561E+06    0.23

k_h= 0.00

Iteration ceiling= 5000

Factor of Safety tolerance= 0.020

trial factor     max displ          iterations
    0.5000      0.1090E+00              2
    1.0000      0.1090E+00             10
    1.5000      0.1377E+00             95
    1.6250      0.1493E+00            143
    1.6875      0.1572E+00            206
    1.7188      0.1666E+00            295
    1.7344      0.1784E+00            535
    1.7500      0.4036E+00           5000

Estimated Factor of Safety =      1.75
```

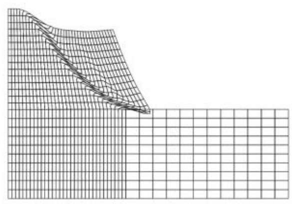
FILE: A75.res (Slope Angle 75 degree)

```
SLOPE64

written by
D.V. Griffiths
Colorado School of Mines
2013
------------------------------------------------------------
Example 1: AMBON 75 deg Slope
w1=   26.00
s1=    7.97
w2=   47.00
h1=   29.11
h2=   25.70

nx1=   32
nx2=   12
ny1=   15
ny2=   10

Group  phi      c      psi    gamma       e          v
  1   26.00  130.00   0.00   22.00  0.2561E+06    0.23

k_h= 0.00

Iteration ceiling= 5000

Factor of Safety tolerance= 0.020

trial factor     max displ          iterations
    0.5000      0.1056E+00              2
    1.0000      0.1060E+00             28
    1.2500      0.1163E+00             71
    1.3750      0.1276E+00            108
    1.4375      0.1398E+00            173
    1.4531      0.1610E+00            897
    1.4688      0.4429E+00           5000

Estimated Factor of Safety =      1.47
```

FILE: A60.dis (Slope Angle 60 degree)



FILE: A75.dis (Slope Angle 75 degree)



FEM SUMMARY of WITHOUT Shotcrete

FEA = Finite Element Analysis (60 & 75 deg only)				
Slope Geometry: Top Slope Angle (Degree) / Bottom Slope Angle(Degree)	60	60	75	75
Material Model / Method: Mohr Coulomb & Finite Element M.	MC & FE (Finite Element)	MC & FE (Finite Element)	MC & FE (Finite Element)	MC & FE (Finite Element)
With Shotrete (Y/N)	NO	NO	NO	NO
Reinforcement (End Anchor) Y/N	NO	NO	NO	NO
Groundwater	NO	NO	NO	NO
Load (Static is 10 kPa), and Eartquake 50% of 0.51 PGA	Static	Earthquake	Static	Earthquake
Safety Factor – LIMIT EQUILIBRIUM – Janbu Corrected	1948	1368	1708	1218
Safety Factor – LIMIT EQUILIBRIUM – Spencer	1870	1419	2.108	1195
Safety Factor – LIMIT EQUILIBRIUM – Morgensten-Price	1871	1399	1777	1207
Safety Factor – by LEM – MEAN from two (Least) Limit Equilibrium Method	1871	1384	1743	1201
Safety Factor – by FEM – Based on FE Approach (Griffiths' Colorado School of Mines)	1750	1310	1470	1160
General Comment	GOOD (no Shotcrete)		NOT GOOD (no shotcrete)	
Indonesian Code for Geotech Requirement (RSNI 2017 & Hynes-Griffin 1984, See Appendix B) >	1.5	1.0	1.5	1.0

APPENDIX E –Tension Crack Analysis

Summary of The TENSION CRACK Analysis

Slope Geometry	Top Slope Angle (Degree)	Bottom Slope Angle(Degree)	Material Model / Method	With Shotrete (Y/N)	Reinforcement (End Anchor) Y/N	Groundwater	Load (Static is 10 kPa), and Eartquake 50% of 0.51 PGA	SF Tension Crack Analysis	SF Mean Earthquake (MC + HB)	General Comment	Indonesian Code for Geotech Requirement (RSNI 2017 & Hynes-Griffin 1984, See Appendix B)
Tension Crack Analysis - with 60 and 75 degree Slope with 20 to 40 dip angle of Joint Rock (if any)	60		Tension Crack (Excel) 20 degree joint	NO	NO	YES (Full)	Static Static	1.994	133%	Good	1.500
			20 degree joint			(Half)	Static	2.778	185%	Good	1.500
	60		Tension Crack (Excel) 30 degree joint	NO	NO	YES (Full)	Static Static	1.248	83%	Not Good	1.500
			30 degree joint			(Half)	Static	1.697	113%	Good	1.500
	60		Tension Crack (Excel) 40 degree joint	NO	NO	YES (Full)	Static Static	0.907	60%	Not Good	1.500
			40 degree joint			(Half)	Static	1.287	86%	Not Good	1.500
	75		Tension Crack (Excel) 20 degree joint	NO	NO	YES (Full)	Static Static	2.050	137%	Good	1.500
			20 degree joint			(Half)	Static	2.968	198%	Good	1.500
	75		Tension Crack (Excel) 30 degree joint	NO	NO	YES (Full)	Static Static	1.259	84%	Not Good	1.500
			30 degree joint			(Half)	Static	1.731	115%	Good	1.500
	75		Tension Crack (Excel) 40 degree joint	NO	NO	YES (Full)	Static Static	0.879	59%	Not Good	1.500
			40 degree joint			(Half)	Static	1.205	80%	Not Good	1.500