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SLOPE STABILITY ASSESSMENT FOR A NEW RESIDENTIAL DEVELOPMENT ON HILLY TERRAIN IN KULIM

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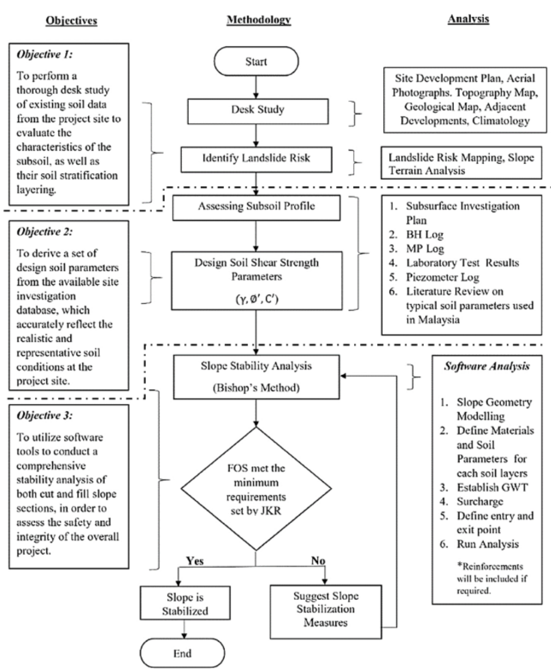
Graphical abstract



Abstract

Step-by-step guidance for new practitioners is essential for understanding the general engineering principles adopted for slope stability analysis within the Malaysian context. In view of this, this paper presents the basic geotechnical assessment conducted for a new residential development on hilly terrain in Kulim. Three objectives were outlined, including desk study based on the secondary data of related maps and reports, designing soil parameters, and assessing the slope stability analysis through numerical modelling for ensuring compliance with the long-term slope stability safety guidelines provided by the Malaysian Public Works Department (JKR). Practical sustainable stabilisation methods were identified, and a feasible foundation design for the development was proposed for the development, considering both cost-effectiveness and minimal environmental impact.

Keywords: Geotechnical Assessment, Factor of Safety, Hilly Terrain, Malaysia, Slope Stability.

1.0 INTRODUCTION

The geotechnical assessment for slope stability using an innovative stochastic framework, considering integrating sophisticated analysis and tools, has steadily drawn the attention of academics in recent times. Despite these significant advances, fundamental knowledge in basic geotechnical assessment applications remains crucial as step-by-step guidance for new practitioners. By having a strong foundation in fundamental geotechnical assessment knowledge, new practitioners can better apply technological advancements in the future, which supports the embracing new technology in construction productivity [9]. In that context, the present study conducted a basic geotechnical assessment to analyse the stability of the slope in the selected critical slope sections for a new residential development on a hilly terrain in Kulim.

The development of Kulim and the expansion of urbanisation in the northern corridor of Kedah and Penang have fuelled significant population growth in Kulim, a city in Kedah [14]. Driven by substantial population pressure, Kulim requires increased residential development to cope with the growing demand. Consequently, the rapid transformation of the city landscape due to heavy construction and deforestation raises concerns about the risk of landslides and soil erosion.

This study pertains to the geotechnical analysis of a well-planned mix development comprising single-story, double-story, and five-storey apartments in Kulim, Kedah. The site is surrounded by industrial facilities, educational institutions, housing, and reserved forests. Figure 1 depicts the site location. Notably, the east-southeast portion is heavily covered by palm plantations.

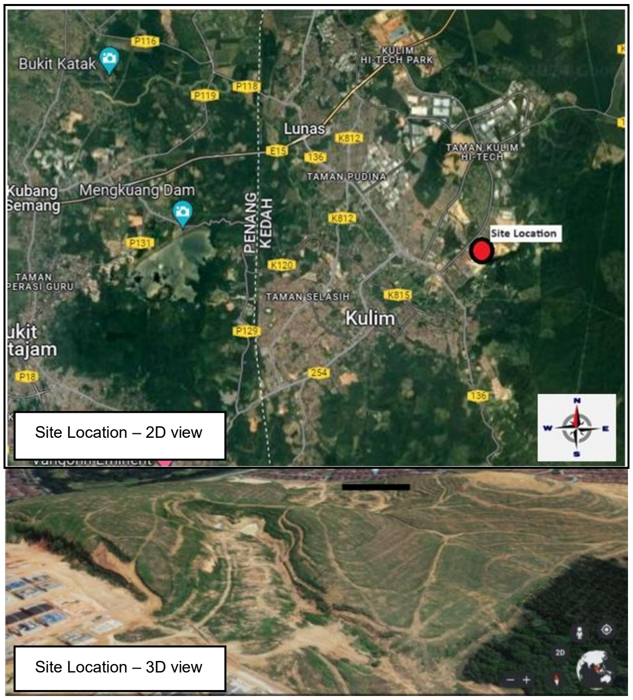


Figure 1 Aeriel photographs [6]

A comprehensive geotechnical assessment is critical for ensuring the project adheres to the stringent safety guidelines outlined by the Malaysian Public Works Department (JKR). This adherence reduces the risk of costly remedial works, such as the need for additional reinforcement for slope stabilisation and infrastructure repairs in the future [7][8][15]. By analysing the detailed findings from the geotechnical investigation of the new development area, this study sheds light on the fundamental engineering practices employed in slope stabilisation methods within the Malaysian context.

Additionally, this paper offers a basic guideline for slope stability assessment for practitioners involved in similar projects, aiming to enhance the understanding and application of slope design principles for sustainable and safe development. For detailed design guidance and reference, practitioners are directed to JKR, Department of Environment (DOE), Minerals and GeoScience Department Malaysia (JMG), local authorities of the city (in this instances, Majlis Perbandaran Kulim), Ministry of Housing and Local Governments (MHLG), Urban and Rural Planning Department (JPBD), The Institution of Engineers Malaysia (IEM), Kumpulan Ikram Sdn. Bhd. (IKRAM), Eurocode 7: Geotechnical Design, and FHWA/NHI 05-123 Soil Slope and Embankment Design.

2.0 SCOPE OF WORK

Geotechnical investigation is the process of developing data on the state of subsurface soil condition and combining it with other relevant information to ascertain the geomaterial parameters needed for the design of the new development [4]. The investigation consisted of the followings, as depicted in Figure 2.

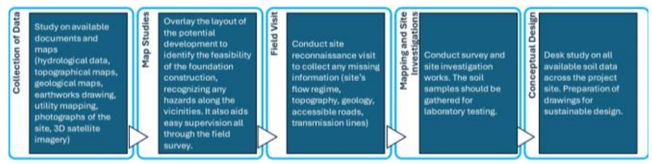


Figure 2 Guidelines for geotechnical investigation works [4][8]

This study referenced the methodologies for the embankment stabilisation design and analysis method, as guided by [8]. The guidelines are as follows:

i. Evaluate the field and laboratory data to interpret the nature and extent of the subsoil condition and their stratification;
ii. Assess the groundwater regime;
iii. Formulate the design soil parameters from the field and laboratory database;
iv. Carry out relevant slope stability analyses on typical cut-and-fill slope sections;
v. Carry out settlement analysis for fill slopes and embankments;
vi. Provide drainages, and close turfing, wherever necessary, and;
vii. Monitor periodically during and after the construction.

The scope of this study is slope stability assessment. Therefore, this study is limited to slope stability analysis and does not cover settlement analysis as it requires further detailed assessment.

3.0 METHODOLOGY

Upon collecting all the information and data from the site, a comprehensive slope stability assessment is conducted. Figure 3 shows a detailed flow chart for conducting the assessment. This flow chart can also serve as a step-by-step guidance for future practitioners.

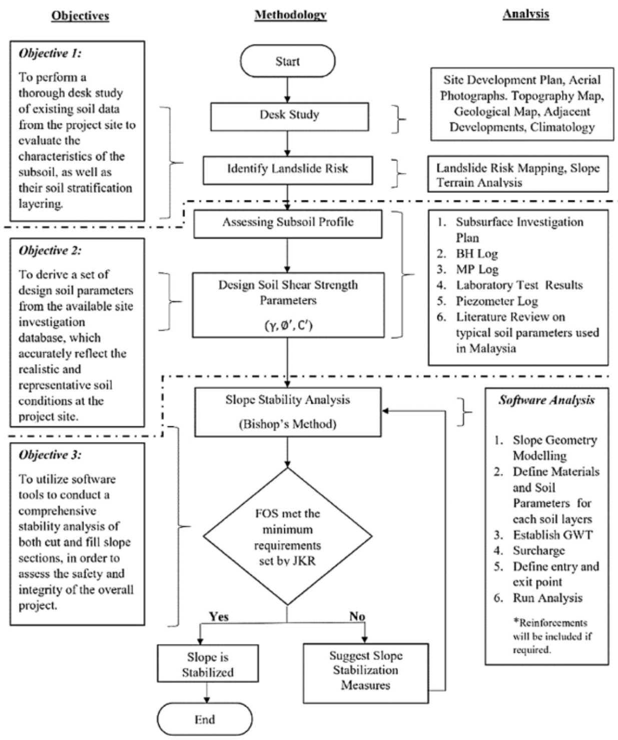


Figure 3 Flow chart for analysing slope stability

4.0 DESK STUDY

Every detail from related maps, plans and other sources is extracted for the assessment purposes. Some of the site information has already been detailed in the Introduction section.

4.1 Geological Mapping

The geological investigation proposes the presence of the Bongsu Granite bedrock underlying beneath the proposed development site [14]. This formation belongs to the Bukit Mertajam-Kulim Granite Formation, as presented in Figure 4. The presence of the bedrock suggests a potentially stable building foundation for high load bearing structures. However, further site investigation works must be carried out to confirm the depth of bedrock for the selection of foundation.

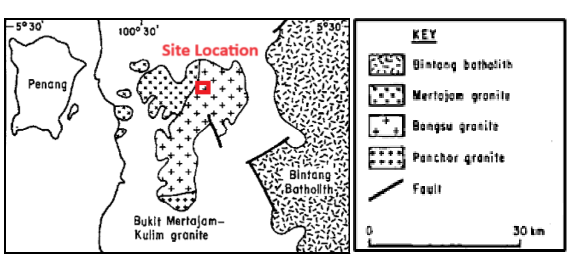


Figure 4 Geological conditions of the proposed development [14]

4.2 Climatology

Infiltration of heavy rainfall is a primary triggering factor for landslides [10]. This infiltration process elevates pore water pressure within the soil, thereby reducing shear strength and promoting slope instability. Consequently, regions with frequent heavy precipitation, such as those with tropical climates like Malaysia, exhibit heightened susceptibility to landslide hazards. According to the National Slope Master Plan, Bandar Kulim is classified as low to medium landslide susceptibility zone [10].

4.3 Topographical Survey and Geological Terrain Mappings

The 145 acres of land has a hilly terrain landscape. A slope terrain analysis was performed to confirm the landslide risk of this case study, aligning with the recommendations of JMG, DOE and JKR. From the survey drawing in Figure 5, the peak of the hill shows an elevation of 83 meters reduced level (mRL) at both the eastern and western sectors, gradually reaching down to 38 mRL. For any development with Class III or Class IV slope, a geological terrain mapping report required to be submitted and reviewed by Geotechnical Accredited Checker registered with Board of Engineers Malaysia [18]. For this development, the geological terrain mapping reports concluded that 97.04% of the site falls under Class I and II, signifying minimal landslide potential. While the remaining 2.96% is classified under Class III and IV, which requires stage-controlled and well-planned earthworks for levelling the platform. Therefore, this proposed development confirmed low risk level for landslide hazards.

4.4 Earthwork Requirement

According to [11], any unsuitable soil with minimum 40 blows per 300mm probe resistance or up to a maximum depth of 3m, whichever is the earlier, is recommended for removal and replacement with suitable and well-compacted materials. In hilly terrain areas, geotechnical engineers often utilise slope engineering to create level building platforms on hillsides to accommodate more buildings, amenities, and infrastructure. The proposed platform formation is the most crucial element, which requires considerations of the following as presented by the [3] and [9]:-

i) Optimizing earthworks to minimize the cost of importing or exporting soil materials;
ii) Optimizing the design for safety to avoid the requirement of very high earth retaining structures;
iii) Effective drainage system with gradual flow gradient;
iv) Reducing any possible risks to proposed development and its surrounding area, and
v) Choosing suitable foundation system for the proposed development

For this case study, the significant difference in the hill elevation proposes to create various platform levels ranging from 57mRL to 63mRL with cut-and-fill works as shown in Figure 5. Therefore, the cut slope required to be designed with a gradient not steeper than 1V:1.5H, while the fill embankment shall not be more than 1V:2H as per the guidelines provided by [18] and [12]. In adherence to the guidelines, both cut and fill slopes for this development adhered to a maximum of 6 flights of soil slope with a maximum of 6m total slope height and a 2m berm width each. The selected critical slope sections are also highlighted in Figure 5.

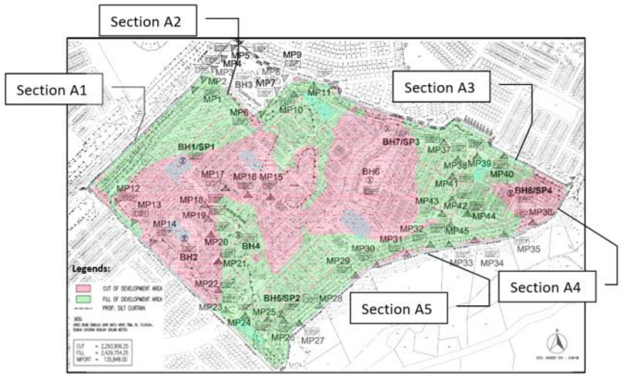


Figure 5 Integration of SI layout and earthwork plans of the proposed development

5.0 FIELD INVESTIGATION

The site investigation (SI) works were conducted in December 2020, and the scope of work consisted of the following:

(i) Eight sample borings (BH1 through BH8) were drilled using a multi-speed water-flushed rotary boring machine with a mast to depths ranging from 19.5m to 36.5m;

(ii) 45 samples were collected from the Mackintosh Probe (MP) Test by driving a rod of 12.7mm diameter with a steel pointer 25.44mm diameter and 60-degree cone using a 4.5kg hammer through a vertical height of 300mm at a maximum depth of 15m below ground level or 400 blows per 30cm, whichever is achieved earlier, and;

(iii) Four standpipes (SP) placed at BH1, BH5, BH7 and BH8.

The SI layout plan is presented in Figure 5. The ASTM-recommended procedures have been performed for the Standard Penetration Test (SPT) [1]. After being sorted, samples were sealed to prevent moisture changes and brought to the laboratory.

The interpreted stratification of the subsurface layering and the location of the groundwater table (GWT) is presented in Figure 6. Based on the borehole assessment, none of the boreholes had encountered bedrock within the investigation depth as suggested by the geological profile. This suggests the depth of bedrock likely exceeds the borehole investigation depth.

The layering distribution based on the MP Tests is presented in Figure 7. The soil strength is then analysed based on JKR's Probe guidelines [5][13]. Any highlighted MP values less than 40 in the layering distribution indicates weak soil strength. From the result, MP7 and MP8 exhibit low shear strength of soil up to a depth of 3m. Based on the subsoil profile in Figure 6, the nearest borehole within the active area identifies cohesive soil deposits extending to a depth of 3 meters.

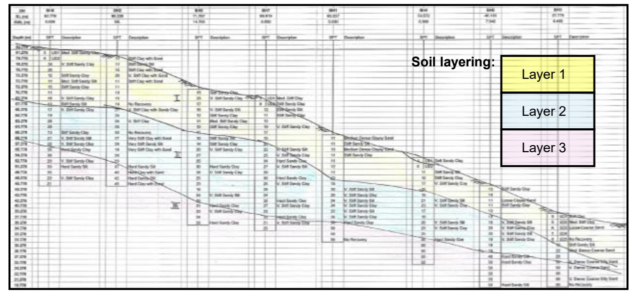
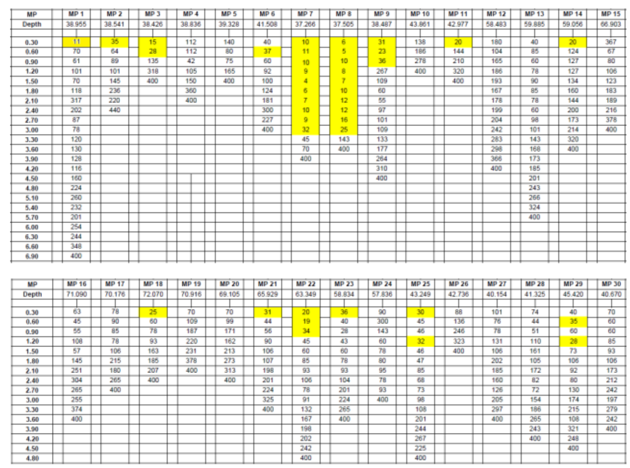


Figure 6 Subsoil profile and GWT level of the proposed development

5.1 Interpretation of Subsoil Condition

Referring to Figure 6, the subsoil layering can be classified into three layers. The relationship between the soil bearing capacity, the depth and thickness of soil layering is tabulated in Table 1. The soil bearing capacity is classified from very light to high load bearing structures utilising the SPT N-value and the empirical relationship recommended by [20].



MP	MP 1	MP 2	MP 3	MP 4	MP 5	MP 6	MP 7	MP 8	MP 9	MP 10	MP 11	MP 12	MP 13	MP 14	MP 15
Depth	38.955	38.541	38.426	38.836	39.328	41.508	37.266	37.505	38.487	43.861	42.977	58.483	59.885	59.056	66.903
0.30	11	35	15	112	140	40	10	6	31	138	20	180	40	20	367
0.60	70	64	28	112	80	37	11	5	23	186	144	104	85	124	67
0.90	61	89	135	42	75	60	10	10	36	278	210	165	60	127	80
1.20	101	101	318	105	165	92	9	8	267	400	320	186	78	127	106
1.50	70	145	400	150	400	100	4	7	109		400	193	90	134	123
1.80	118	236		360		124	6	10	60			167	85	160	183
2.10	317	220		400		181	7	12	55			178	78	144	189
2.40	202	440				300	10	12	97			199	60	200	216
2.70	87					227	9	16	101			204	98	173	378
3.00	78					400	32	25	109			242	101	214	400
3.30	120						45	143	133			283	143	320	
3.60	130						70	400	177			298	168	400	
3.90	128						400		264			366	173		
4.20	116								310			400	185		
4.50	160								400				201		
4.80	224												243		
5.10	260												266		
5.40	232												324		
5.70	201												400		
6.00	254														
6.30	244														
6.60	348														
6.90	400														

MP	MP 16	MP 17	MP 18	MP 19	MP 20	MP 21	MP 22	MP 23	MP 24	MP 25	MP 26	MP 27	MP 28	MP 29	MP 30
Depth	71.090	70.176	72.070	70.916	69.105	65.929	63.349	58.834	57.836	43.249	42.736	40.154	41.325	45.420	40.670
0.30	63	78	25	70	70	31	20	36	90	30	88	101	74	40	70
0.60	45	90	60	109	99	44	19	40	300	45	136	76	44	35	60
0.90	55	85	78	187	171	56	34	28	143	46	246	78	51	60	60
1.20	108	78	93	220	162	90	45	43	60	32	323	131	110	28	85
1.50	57	106	163	231	213	106	60	60	78	46	400	106	161	73	93
1.80	145	215	185	378	273	107	85	78	80	47		202	105	106	106
2.10	251	180	207	400	313	198	93	93	95	85		185	172	92	173
2.40	304	265	400		400	201	106	104	78	68		160	82	80	212
2.70	265	400				224	78	201	93	73		126	72	130	242
3.00	255					325	91	224	400	98		205	154	174	197
3.30	374					400	132	265		108		297	186	215	279
3.60	400						167	400		201		400	265	108	242
3.90							198			244			243	321	400
4.20							202			267			400	248	
4.50							242			225				400	
4.80							400			400					

Figure 7 Layering distribution based on the MP test

Table 1 General subsoil stratification at the proposed development

Layer	Soil Description	Soil Thickness (m)	SPT N-value (blows per 300mm penetration)	Soil bearing capacity classification [20]
I	Med. Stiff to Stiff Sandy Silt / Loose to Medium Dense Silty Sand	7.5 ~ 15.0	4 – 15	Very light to low load bearing structures
II	Very Stiff Sandy Silt / Sandy Clay	3.0 ~ 18.0	16 – 29	Moderate load bearing structures
III	Hard Sandy Clay / Hard Sandy Silt / Very Dense Silty Sand	Below	>30	High load bearing structures

The summary in Table 1 proves the bearing capacity of soil increases with the SPT N-value. The GWT levels reported in Figure 6 range from 0.4m to 15m from the existing ground level. A conservative groundwater level of about 5m from the highest platform can be considered for slope analyses.

6.0 LABORATORY TESTING

The design of cut slopes prioritizes long-term stability (drained condition with effective stresses, σ') over short-term stability (undrained condition with total stresses, σ). Typically, in Malaysia, the shear strength of the subsoil condition is evaluated through triaxial and direct shear tests [2]. Therefore, Consolidated Isotropically Undrained (CIU) Triaxial tests and Shear Box Tests were performed by soil lab specialists on selected undisturbed samples to determine the shear strength parameters of the soil. The results of the laboratory test are presented in Table 2.

Table 2 Results of laboratory test performed by soil lab specialist

BH No.	Sample No.	Depth (m)	CIU Triaxial Test – 38mm				Shear Box Test	
			Total Stress		Effective Stress			
			c (kPa)	Ø (°)	c (kPa)	Ø (°)	c (kPa)	Ø (°)
BH3	UD1	3.50 to 4.50	27	18	7	35	-	-
BH4	UD1	2.00 to 3.00	17	14	3.5	34	-	-
BH6	UD1	2.00 to 3.00	20.5	15	8	32	0.5	34
BH7	UD1	2.00 to 3.00	18	16	5	36	0.5	34

where c is the cohesion in kPa and Ø is soil's internal friction angle in degree.

6.1 Design Soil Parameters

For slope stability analysis, a significant engineering judgment is applied to facilitate geotechnical evaluation, contributing to the selection of conservative parametric values for a safe and efficient design as recommended [19]. The lowest c' value is adopted to ensure a conservative design. From the result tabulated in Table 2, the lowest c' is 3.5kPa (BH4). Meanwhile, for the angle of internal friction, an average value is adopted, therefore, Ø' is 34^{o}. Thereby, with a significant engineering judgement, the following soil parameters were adopted incorporated into slope stability analyses to evaluate the factor of safety against failure:

LAYER I : γ = 18kN/m^3, Ø' = 31°, c' = 2 kPa
LAYER II : γ = 19 kN/m^3, Ø' = 34°, c' = 3 kPa
LAYER III : γ = 20 kN/m^3, Ø' = 36°, c' = 5 kPa
FILL MATERIAL: γ = 19 kN/m^3, Ø' =30°, c' = 1 kPa

7.0 SLOPE STABILITY ANALYSIS

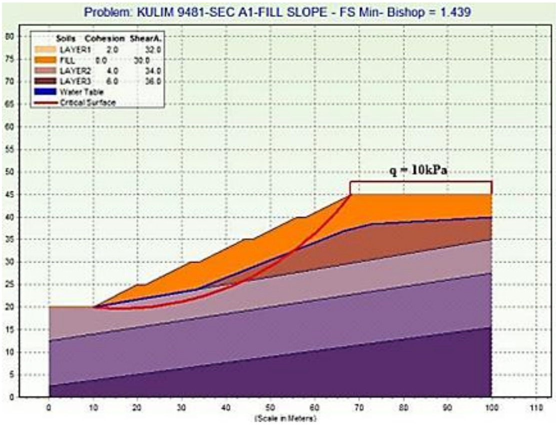
For five slope sections, identified as Section A1 through Section A5 in Figure 5, represent the critical slopes spanning various ranges of critical slope heights. Slope stability assessment incorporating the limit equilibrium modelling (LEM) technique using "StablPro – Ensoft Inc." were performed to assess the safety factor of the suggested cut-and-fill slopes under drained conditions for long-term stability.

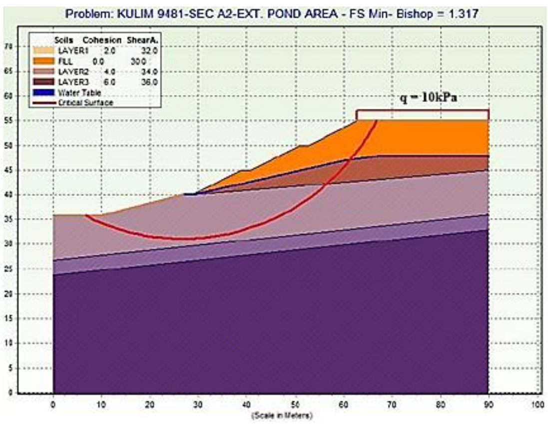
LEM has historically been the primary method for estimating slope stability. However, one of the limitations of the LEM technique is its inability to model the non-linear stress-strain behaviour of soil materials, unlike Finite Element Modelling (FEM), which can provide a more detailed picture of stability-based deformations. Nevertheless, the traditional LEM appeared to produce accurate and reliable results as confirmed by [16] where the FOS differences between FEM and LEM results are small and it can be used as a preliminary step to validate the results of complex numerical models. Additionally, LEM relies on assumptions regarding slice side forces. To identify the most critical slip surface failure, the Bishop's Circular Slip Method was employed. A surcharge of 10kPa has been considered in the analysis. The results of the analysis are presented in Table 3 and Figure 8. The results are then compared with the minimum requirement for FOS as per the guidelines provided by [8] in Table 4.

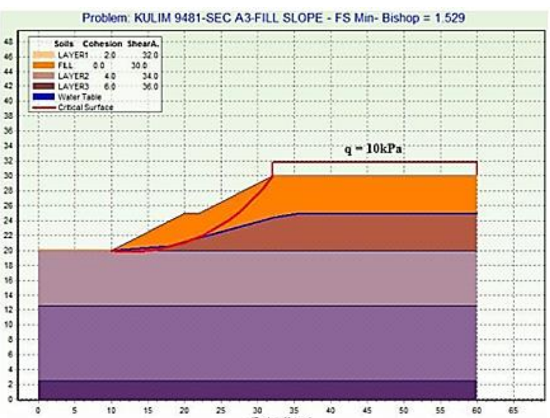
Table 3 Results of slope stability analysis

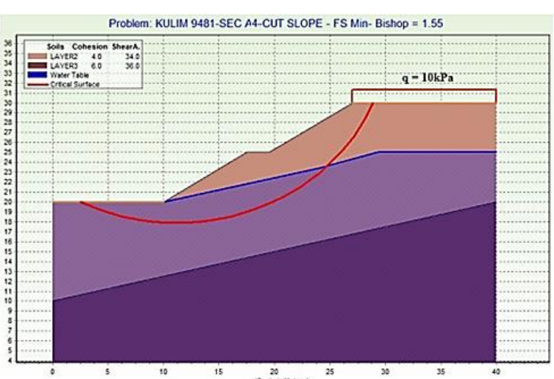
Mode of failure - Toe failure			
Critical slope sections	**Description**	**Closest Reference Borehole**	**FOS achieved against failure**
Section A1-A1	• Fill slope • Maximum 5 flights • 2m wide berm • 1V:2H slope gradient	BH1	1.439 (Stable)
Section A2-A2	• Fill slope abutting existing pond • Maximum 4 flights • 2m wide berm • Slope gradients: ✓ 1V:2H for all flights except the first ✓ 1V:4H for the first flight at the slope toe	BH3	1.317 (Stable)
Section A3-A3	• Fill Slope • Maximum 2 flights • 2m wide berm • 1V:2H slope gradient	BH8	1.529 (Stable)
Section A4-A4	• Cut Slope • Maximum 2 flights • 2m wide berm • 1V:1.5H slope gradient	BH8	1.550 (Stable)
Section A5-A5	• Fill Slope • Maximum 6 flights • 2m wide berm • 1V:2H slope gradient	BH6 & BH8	1.387 (Stable)

Note: Soil slope designed with maximum 6 flights (maximum of 5m per flight and a 2m berm width each). FOS achieved more than 1.3 for the proposed unreinforced soil as per the slope design requirement by JKR. Therefore, the slope sections are deemed stable.









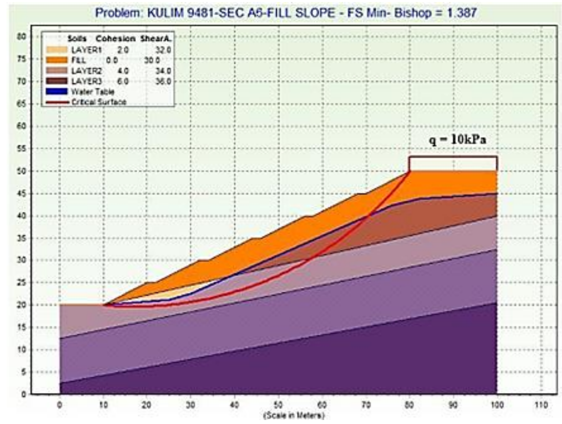


Figure 8 Most critical slip surface circle for all critical sections

Table 4 JKR guidelines for designing slope in Malaysia [8]

Guidelines for Slope Design

SOME TYPICAL GEOTECHNICAL DESIGN CRITERIA FOR SLOPES DESIGN

DESIGN COMPONENT	MODE OF FAILURE	MINIMUM FACTOR OF SAFETY	MAXIMUM PERMISSIBLE MOVEMENTS		
			VERTICAL	LATERAL	DIFFERENTIAL
1. Unreinforced Slopes	1.1 Local & Global Stability (cut & fill slopes) 1.2 Bearing (fill)	1.3 2.0	Analysis should be according to GEOTECHNICAL MANUAL FOR SLOPES (1984), GEO Hong Kong		
2. Reinforced or Treated Slopes (not on soft ground)	2.1 Local & Global Stability (cut & fill slopes) 2.2 Bearing (fill)	1.5 1.5			
3. Permanent Anchors	3.1 Tensile Resistance 3.2 Resistance at Soil Grout Interface 3.3 Creep/Corrosion	2.0 3.0	Geo Spec 1 (1989), GEO Hong Kong BS 8081		
4. Rigid Retaining Structures	4.1 Overturning 4.2 Sliding 4.3 Overall Stability 4.4 Bearing	2.0 1.5 1.5 2.0	15mm along face of wall Geoguide 1 (1983), GEO Hong Kong	15mm along face of wall	1 : 150 along face of wall
5. Reinforced Fill Walls/Structures	External Stability Internal Stability	BS 8006	± 5mm per metre height	± 15mm from reference alignment	1 : 100 along face of wall
6. Individual Foundation Piles (mainly under axial loads)	6.1 Shaft Resistance	2.0	12mm along axis of pile at pile head at design load. 38mm or 10% pile size at pile head at twice design load.		
	6.2 Base Resistance	2.0	BS 8004		
7. Individual Foundation Loads (mainly under lateral & bending loads perpendicular to axis of pile)	Ultimate Lateral Resistance	2.5	12mm along axis of pile at pile head at design load. BS 8004	12mm perpendicular to axis of pile at design load	
8. Pile Group	Block Bearing Capacity	2.0	12mm at Working Load BS 8004		
9. Piles as Retaining Structures	As for 4, 6 & 7 above	As for individual foundation piles	As 4 above for rigid retaining structures BS 8004		
10. Embankment on Soft Ground	10.1 Bearing (short term) 10.2 Local & Global Slope Stability (long term)	1.4 1.2	7 years post construction settlement : (i) within 10m from bridge approach < 100mm (ii) road < 250mm		

Based on Table 4, a minimum FOS of 1.30 is required for proposing unreinforced cut and fill slopes under global stability mode. In this regard, the computed safety factors in Table 3 and Figure 8 are deemed adequate and compliance to JKR guidelines. Hence, no additional reinforcements are required.

8.0 SLOPE MAINTENANCE RECOMMENDATIONS

Despite having adequate FOS for the proposed slope sections, additional measures are required to ensure better stability of slope. The additional measures are presented in Figure 9.

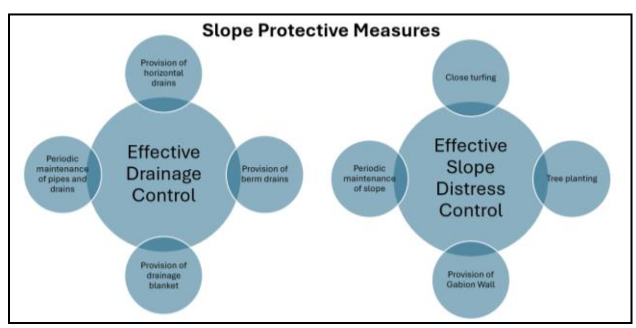


Figure 9 Slope Protective Measures [8]

9.0 FOUNDATION DESIGN CONSIDERATIONS

Considering the condition of the subsoil and proposed earthworks, for low load-bearing structures such as single-story and double-story houses, the construction method shall involve excavating the ground to the required depth and building on shallow foundations. Meanwhile, the five-storey apartment shall be on a precast piling system as the less compressible strata is situated at 25m depth below the ground. Further confirmation is required for the fill areas, and it shall be decided based on the post-earthworks confirmatory soil investigation.

10.0 CONCLUSION

In general, the basic geotechnical assessment for a new residential development on hilly terrain in Kulim was conducted and believed to have provided a clear step-by-step guidance for the new practitioners.

The site consists of slopes ranging from Class I to Class IV, necessitating staged and controlled earthworks to create levelled platforms. Despite the site is underlain by the Mertajam Granite Formation, no bedrock was found upto the borehole depth of 42m. In addition, the subsoil conditions are generally comprised of sandy clay, sandy silt and sand.

The field and laboratory test results were analysed to derive the soil parameters used for design purposes. For the sections involving both cut and fill slopes, it is apparent that the subsoil conditions can withstand a surcharge load of 10kPa without any additional reinforcement, achieving a safety factor of 1.3 in compliance with the JKR's slope design guidelines. Several additional measures have been recommended for ensuring better stability of the slope.

Based on the preliminary results, shallow foundations are recommended for low-load bearing structures, while deep foundations are suggested for moderate to high-load bearing structures. It is also highly recommended that all geotechnical works are implemented with high-quality workmanship to achieve stable condition of slope and building platform.

11.0 LIMITATION AND RECOMMENDATIONS

The geotechnical analysis was limited to slope stability analysis only. The settlement of the slope design was not considered as it requires a detailed analysis using appropriate software. A separate comparison settlement analysis study can be approached in the future by comparing the computed settlement value and the post-construction settlement value. The recommended foundation design shall also be confirmed based on the post-earthworks confirmatory soil investigation.

Acknowledgement

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Conflicts of Interest

The author(s) declare(s) that there is no conflict of interest regarding the publication of this paper

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