# CHAPTER VIII

# ROCK PORTAL SLOPE STABILITY ANALYSIS

## 8.1 Rock Slope Stability Analysis

The field investigation of rock slope stability condition was carried out mainly on the slope faces of upstream and downstream portals, which would be further excavated for the water diversion purposes, and on some permanent slope faces which are still exposed after the completion of the diversion tunnel construction. This study was done using the CSIR Geomechanics Classification and some graphical methods of rock slope stability analysis.

### 8.1.1 CSIR Geomechanics Classification

Steffen (1976) was among the first who applied the Geomechanics Classification to the rock slopes. He classified 35 slopes of which 20 had failed by using the Geomechanics Classified-cation of Bieniawski (1973). The output from this classification is the average values of rock-mass cohesion and friction data which were grouped into five rock mass classes. He then calculated the factors of safey by using the analysis charts designed by Hoek and Bray (1974).

However, the method seems inappreciate since it does not include the discontinuity attitudes for adjusting the basic RMR value in a case of slope. Recently, Luangpitakchumpol (1982) had proposed an adjustment of the RMR values for the favourable joint orientation which affect the slope stability. His proposal appears usable in the slope stability analysis at Khao Laem damsite (Table 8.1).

In the present study, the methods of the portal slope stability analysis are those of Bieniawski (1979) and Luangpitakchumpol (1982). The study results are summarized in Tables 8.2 and 8.3.

### 8.1.1 Graphical Analysis of Stability

The design and stability analysis of the rock slope is associated with the geological structures which directly control the types of slope failure. In the analysis of rock slope stability, the orientation of these geological structures, especially the discontinuities and the shearing resistance along them is governed. Four basic modes of failures are considered in this study, i.e. the circular failure, plane failure, wedge failure, and toppling failure. The graphical analysis methods recently developed by Markland (1972), and Hendron et al. (1971) are discussed below.

#### 8.1.2.1 Markland's Method

Hoek and Bray (1981) had simplified the diagram originally developed by Markland (1972) as shown in Figure 8.1. This diagram demonstrates four main modes of slope failures

Table 8.1 Assessment of joint orientation favourability upon stability of slope (after Luangpitakchumpol, 1982).

| Dipping to the same side as slope | | | | Dipping to opposite side of slope | |
|---|---|---|---|---|---|
| $\phi < \delta < \alpha$ | | $\delta < \phi < \alpha$ | $\delta > \alpha$ | $\beta = 0^0 - 45^0$ | $\beta = 45^0 - 90^0$ |
| $\beta = 0^0 - 45^0$ | $\beta = 45^0 - 90^0$ | | | | |
| Very Unfavourable | Unfavourable | Fair | Very Favourable | Very Favourable | Favourable |

where as,
$\delta$ = Dip Angle of Joint
$\alpha$ = Angle of Slope
$\beta$ = Angle between Joint Strike and Slope Line
$\phi$ = Interfrictional Angle

Note:
1. Water pressure is not considered.
2. This table is base on, the concept of slope failure criteria by MARKLAND (1972) & HOCKING (1976) in HOEK & BRAY (1977) and by GOODMAN (1964) & JOHN (1968) in HENDRON (1980) especially for wedge failure. (The most critical one is "very unfavourable". If the failure occures along the intersection of two planes, both will be "unfavourable")

Table8.2 The slope information and results of the analyses by markland's (1972) and Hendron et al.'s technique

| Subarea | Rock Type | Slope Orientation Strike (degree) | Slope Orientation Dip (degree) | Slope Height (m.) | Slope Width (m.) | RMR System | In-Situ Condition | Markland Analysis | Potential Sliding Direction Plung | Potential Sliding Direction Trend | Failure Characteristic | Safety Factor | Stable Slope Orientation Strike ≥ | Stable Slope Orientation Dip ≤ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| U-1 | Gwke | S57°W | 66° | 11-14 | 34 | Very Poor | Rockslides | Unstable | 40°<br>42°<br>50°<br>42°<br>42° | N60°W<br>N60°W<br>N64°W<br>N70°W<br>N85°W | Plane<br>Plane<br>Wedge<br>Wedge<br>Plane | 0.68<br>0.68<br>1.34<br>0.69<br>0.66 | S57°W | 44° |
| U-2 | ,, | S87°W | 61° | 10-14 | 30 | Very Poor | Rockslides | Unstable | 39°<br>38° | N34°W<br>N24°W | Plane<br>Plane | 0.95<br>1.07 | S87°W | 40° |
| U-3 | ,, | S81°W | 59° | 13-15 | 33 | Fair | Stable | | | | | | | |
| U-4 | ,, | N36°W | 78° | 13-15 | 40 | Fair | Rockslides | Unstable | 52°<br>50° | S84°E<br>S80°E | Plane<br>Plane | 0.55<br>0.51 | N36°W | 61° |
| U-5 | ,, | N33°W | 55° | 10-13 | 9 | Poor | Rockslides | Unstable | 43°<br>41°<br>47° | S60°W<br>S38°W<br>S24°W | Plane<br>Wedge<br>Wedge | 0.75<br>0.77<br>0.78 | S33°E | 42° |
| U-6 | Sark | S04°W | 70° | 10-15 | 30 | Good | Stable | Stable | | | | | | |
| U-7 | Gwke | S65°W | 66° | 13-17 | 72 | Very Poor | Rockfalls | Unstable | 57° | N44°W | Wedge | 0.81 | S65°W | 57° |

Note : Upstream portal slopes

Gwke = pebbly graywackes to pebbly mudstones ; Sark = subarkosic sandstones

**Table 8.3** The slope information and results of the analyses by Markland's (1972) and Hendron et al.'s (1971) technique

| Subarea | Rock Type | Slope Orientation | | Slope | | RMR System | In-Situ Condition | Markland Analysis | Potential Sliding Direction | | Failure Characteristic | Safety Factor | Stable S Orientation | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Strike | Dip | Height (m.) | Width (m.) | | | | Plunge | Trend | | | Strike ≥ | Dip ≤ |
| D-1 | Gwke | N48°W | 66° | 8–12 | 20 | Good | Stable | Stable | | | | | | |
| D-2 | ,, | N56°W | 67° | 6–7 | 11 | Good | Stable | Stable | | | | | | |
| D-3 | ,, | N33°W | 62° | 6–7 | 19 | Fair | Rock | Unstable | 38°<br>44° | N16°E<br>N50°E | Plane<br>Plane | 1.33<br>0.70 | N33°W | 44° |
| D-4 | ,, | N28°W | 67° | 4–5 | 13 | Fair | Stable | Stable | | | | | | |
| D-5 | ,, | N28°W | 67° | 7–10 | 12 | Fair | Stable | Stable | | | | | | |
| D-6 | ,, | N34°W | 63° | 12–15 | 30 | Poor | Rockslides | Unstable | 61°<br>62° | N29°E<br>N45°E | Plane<br>Plane | 0.49<br>0.51 | N34°W | 62° |
| D-7 | ,, | N14°W | 76° | 8–11 | 20 | Poor | Rockslides | Unstable | 44°<br>42° | S37°E<br>S46°E | Plane<br>Plane | 0.79 | N14°W | 53° |
| D-8 | ,, | N27°W | 62° | 14 | 30 | Good | Stable | Stable | | | | | | |
| D-9 | ,, | N40°W | 62° | 14 | 25 | Poor–Fair | Rockfalls | Unstable | 38° | S80°E | Plane | 0.80 | N40°W | 52° |
| D-10 | ,, | N49°W | 62° | 14 | 18 | Fair–Good | Rockfalls | Stable | | | | | | |
| D-11 | ,, | N63°W | 70° | 7–10 | 10 | Fair | Stable | Stable | | | | | | |

Note : Downstream portal slopes ; Gwke = pebbly graywackes to pebbly mudstones and a stereonet-plot of the structural conditions likely to create such failures. IN this analysis method, the cut face of slope must be included in the sterenet-plot because the sliding occurs toward the free cut face.

Markland's (1972) test was designed to establish a potential wedge failure with the sliding down the line of intersection of two planar discontinuities as illustrated in Figure 8.1c. The stability of slope, illustrated as the factor of safety, depends on the plunge of the line of intersection, shearing resistance of the discontinuity surfaces, and the geometry of wedge. In a case of plane failure based on limit equilibrium concept, the sliding may occur when the dip of a plane of weakness exceeds the internal friction angle ($\phi$) as the shearing resistance on this plane depends on the friction only. In a case of wedge failure, when the plunge of the intersection line exceeds the friction angle of the rock substance, the sliding occurs. The use of Markland's method was summarized by Hoek and Bray (1981) as illustrated in Figure 8.2. It could be concluded that the slope is potentially undtable when a point defining the line of intersection of the two planes falls within the "day light" area enclosed between the great circle defining the slope face and the circle defined by the angle of friction ($\phi$) as illustrated in Figure 8.2b. It should also be note that both the lower hemisphere equal-area and lower-hemisphere equal-angle stereographic net plots are applicble in this analysis method.

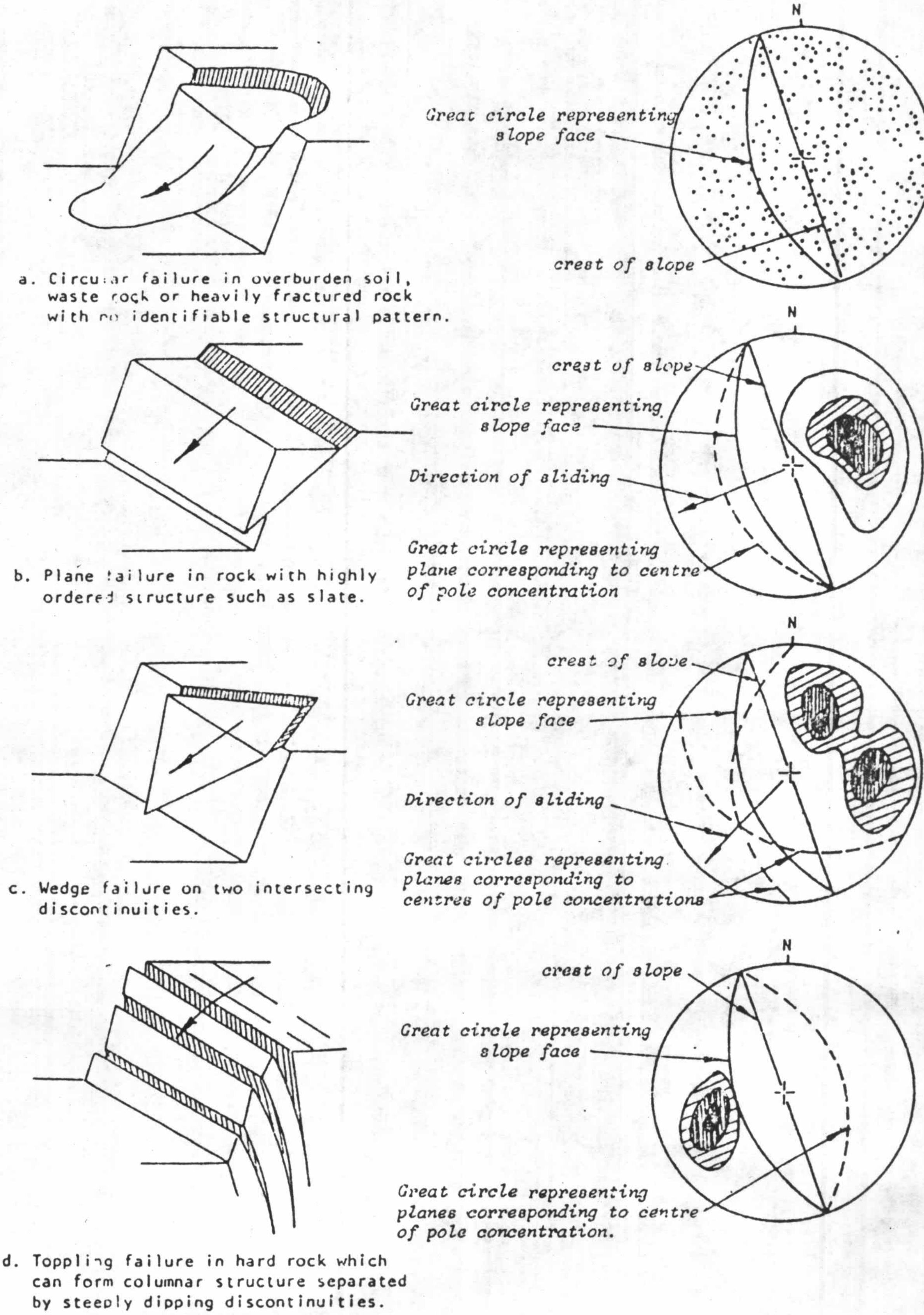
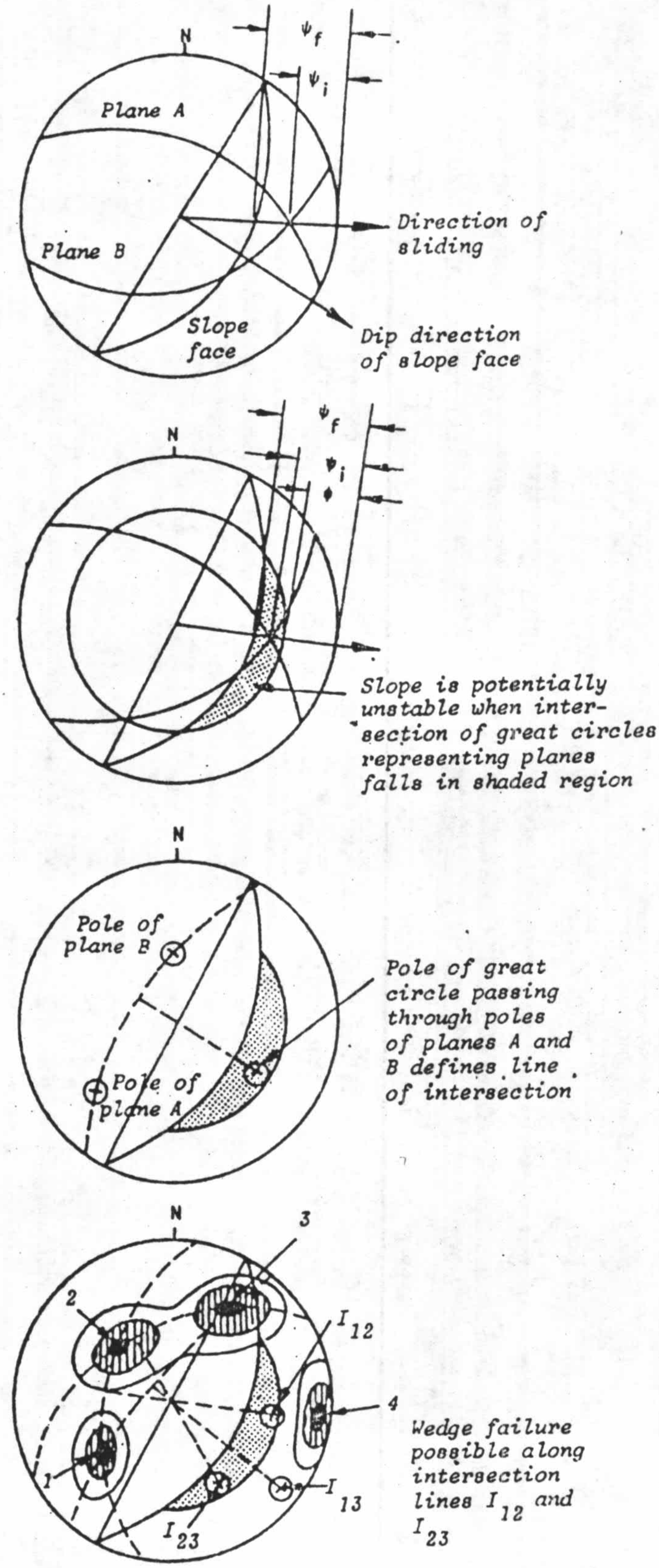



Figure 8.1 Recognition diagram for different types of slope failure and lower hemisphere equal area projection (after Hoek and Bray, 1981).




a: Sliding along the line of intersection of planes A and B is possible when the plunge of this line is less than the dip of the slope face, measured in the direction of sliding, ie

$$\psi_f > \psi_i$$

b : Sliding is assumed to occur when the plunge of the line of intersection exceeds the angle of friction, ie

$$\psi_f > \psi_i > \phi$$

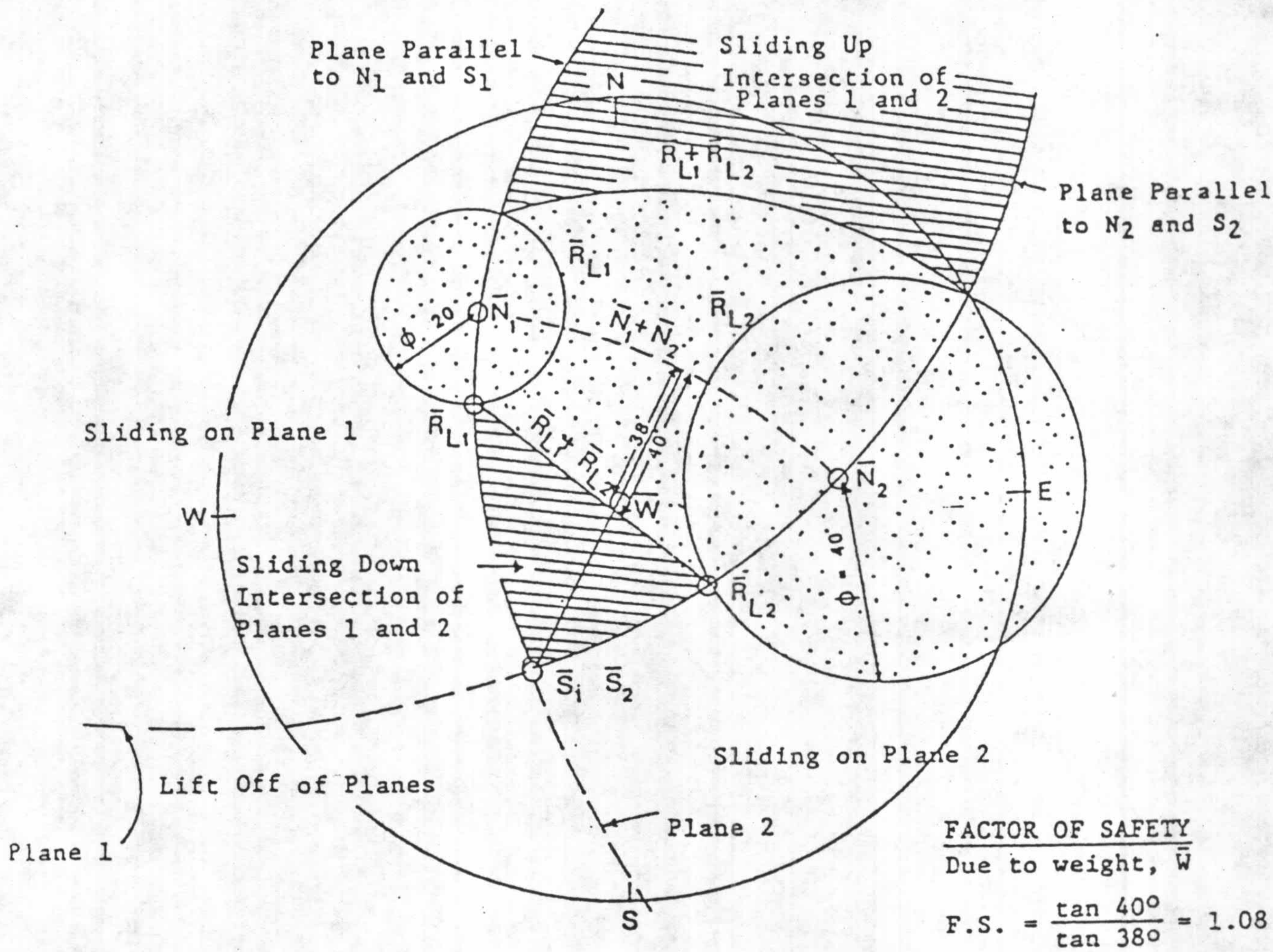
c : Representation of planes by their poles and determination of the line of intersection of the planes by the pole of the great circle which passes through their poles.

d : Preliminary evaluation of the stability of a 50° slope in a rock mass with 4 sets of structural discontinuities.

Figure 8.2 Markland's method of rock slope stability analysis (after Hoek and Bray, 1981).

8.1.2.2 Hendron's Method

Hendron et al. (1971) designed a graphical method to analyse the slope stability in the rock masses. In this method the lower hemisphere equal-angle net is used. The orientation of the force vector which makes an angle with the normal to the potential failure planes greater than the angle of the maximum resisting reaction causes a sliding. The stability delineate is divided into two distinctive parts. In the first part, the orientation of the maximum resisting reaction on the potential failure planes is plotted on the stereonet. The zones of stability and instability can be outlined on the stereonet by considering the orientation of the reactions on the potential sliding planes. The second part involves a determination of the orientation of the resultant driving force action on the wedge. This force includes the weigth of the wedge, acceleration force, uplift water pressure, and other driving forces. Graphical vector directions are used in conjunction with the stereonet to determine the orientation of the resultant force vector. If the orientation of the resultant driving force falls within the zone of stability on the stereogram, the wedge is stable; if not, the wedge failure is possible. The safety factor of this method can be readily determined from the ratio of the tangent of the angle of the maximum resisting force to that of the resultant driving force. The detailed sliding direction and factor of safety on two frictional planes is illustrated as an example in Figure 8.3.




**FACTOR OF SAFETY**

Due to weight, $\bar{W}$

$$F.S. = \frac{\tan 40^o}{\tan 38^o} = 1.08$$

$\bar{N}_1$, $\bar{N}_2$ are normal forces of plane 1 and plane 2 located at the poles of planes

$\bar{S}_1$, $\bar{S}_2$ are maximum shear forces of plane 1 and plane 2, plot on the stereogram at the same point as does the line of intersection of plane 1 and plane 2

$\bar{R}_{L1}$,$\bar{R}_{L2}$ are reaction forces at failure (summation of the normal forces $\bar{N}_1$, $\bar{N}_2$ and maximum shear forces $\bar{S}_1$, $\bar{S}_2$), $\bar{R}_{L1}$ located where a great circle through $\bar{N}_1$ and $\bar{S}_1$ intersects the friction cone of plane 1.

$\bar{N}_1$+$\bar{N}_2$ is a great circle of $\bar{N}_1$, $\bar{N}_2$

$\bar{S}_1$+$\bar{S}_2$ is a great circle of $\bar{S}_1$, $\bar{S}_2$

Figure 8.3 An example of Hendron et al.'s technique for wedge failure analysis

## 8.2 Collecting and Recording Data on Slope Stability Condition

The collecting and recording data, besides the previously mentional geological logging and the rock mass classification, are described below.

### 8.2.1 Plane of Weakness

The planes of weakness are related to the sets of discontinuities, some of which can be a potential failure surface. The geological data in the terms of dip angle and strike direction were analysed using the equal-area stereographic projection, 1 - percent area counting and contouring in order to obtain the preferred orientation of the discontinuity planes which are potential to fail.

### 8.2.2 Surface Roughness

The surface roughness of the discontinuities including their condition and the wall strength were measured. The existing normal stress were estimated. The results were used to predict the shear strength of rough joints as given by equation in Figure 8.4 and 8.5. They were further used in the analyses of stability condition.

### 8.2.3 Slope Parameters

The stability condition of each slope face was d described. The condition consists the slope orientation, inclination, slope height, seepage of groundwater, effect of rainfalls, the potential failure surface and degree of weathering. The

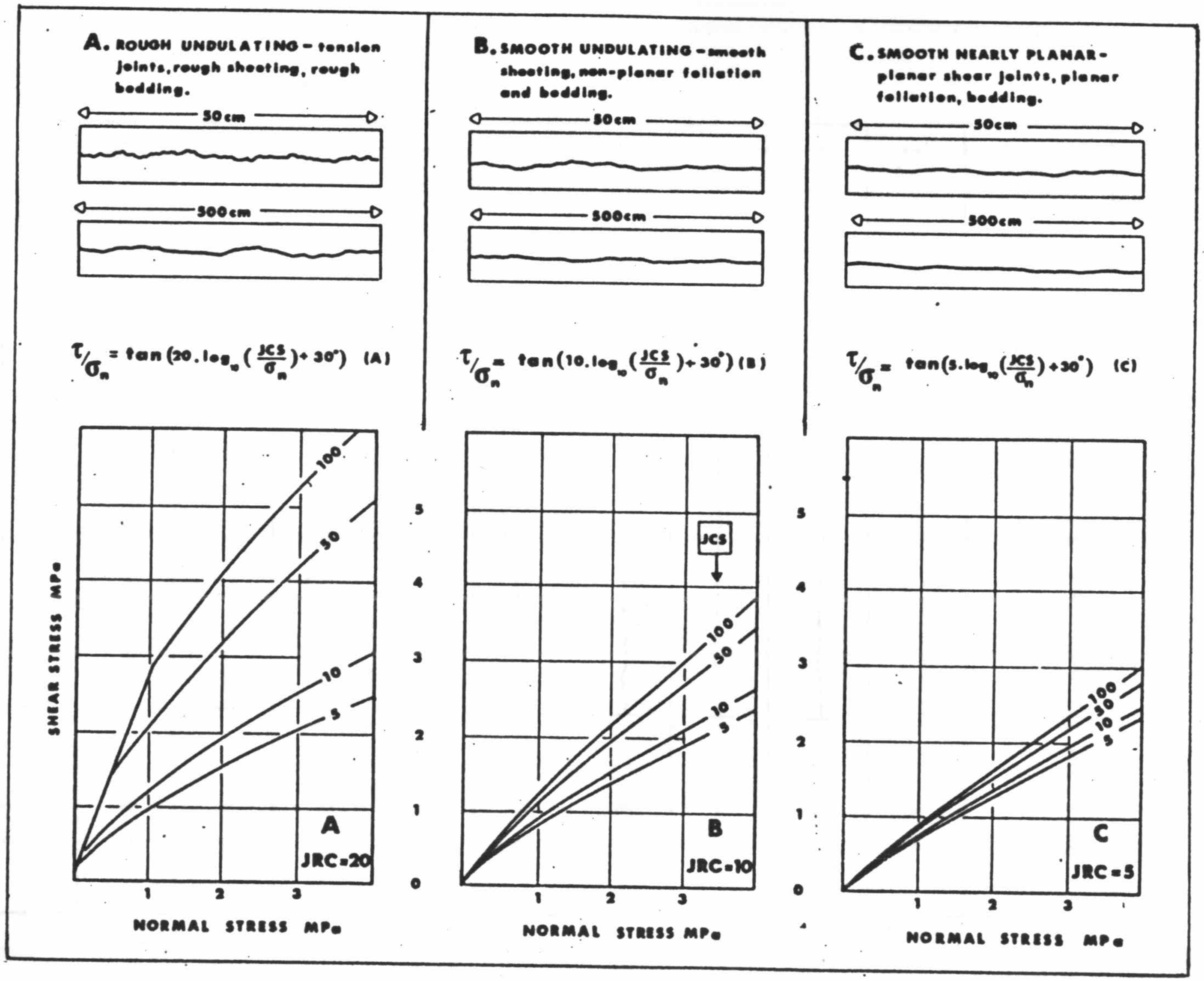



Figure 8.4 A method of estimating peak shear strength from roughness profiles. Each curve is numbered with the appropriate JCS value (units of MPa). The roughness profiles are intended as an approximate huide to the appropriate JRC values 20.10 and 5. Completely smooth planar joints have JRC = 0. (after ISRM, 1981).

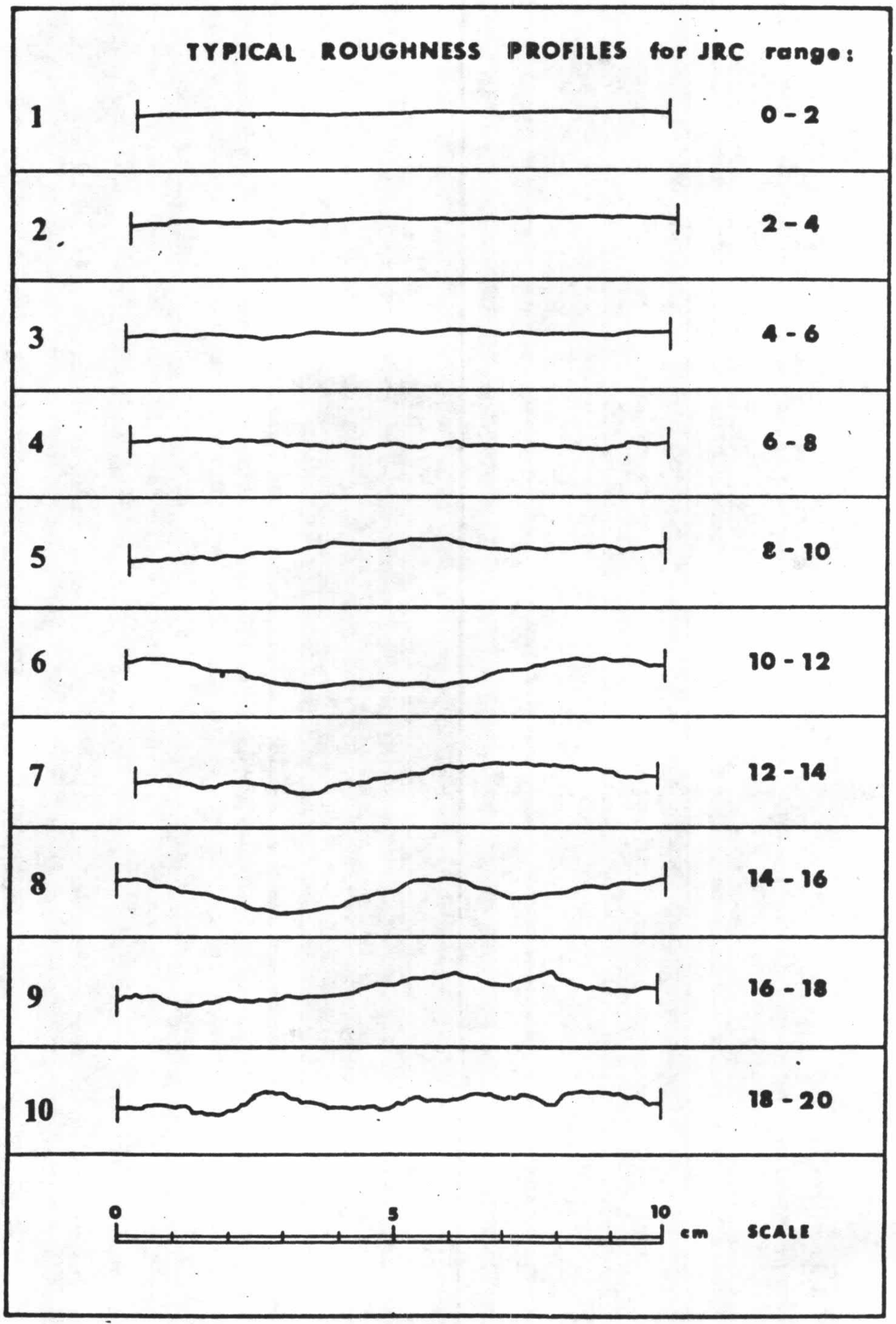



Figure 8.5 Roughness profiles and corresponding range of JRC values associated with each one.

existing evidences of the slope failure was noted, so was the existing excavation design. These parameters would be used in the stability analyses. Then the recommendation to improve the excavation design would be given.

### 8.2.4 Sliding Plane and Direction

The potential failure surface, the residual friction angle, the orientation and inclination of slope face were to predict the type of potential failures and their sliding direction using Markland's method. The friction angles considered in the analyses were that of the potential failure surfaces. In a case of wedge failure, the friction angle of each plane bounded was plotted and the minimum value was used in the analyses.

### 8.2.5 Safety Factor

If the result of the stability analysis using Markland's method suggests an unstable condition, the factor of safety was further calculated using Hendron's (1971) graphical analysis method.

## 8.3 Results and Discussions of Portals Discontinuities

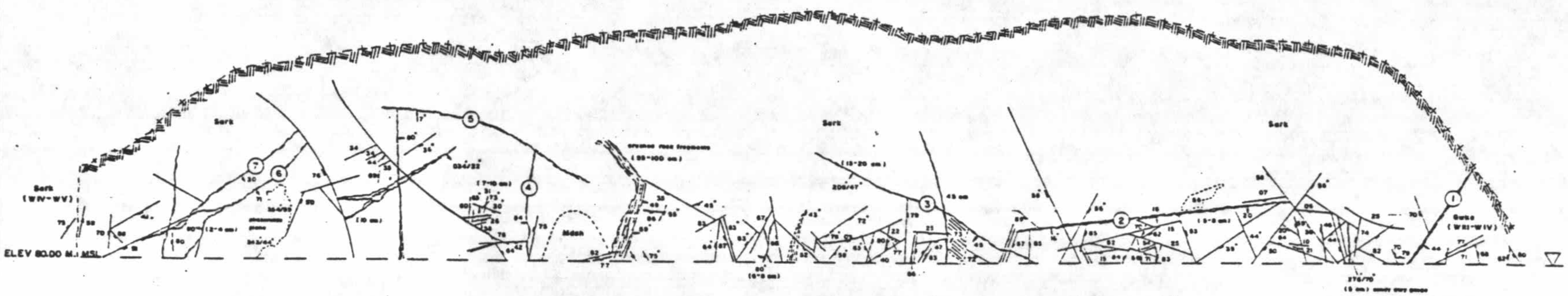
The excavation of the diversion intake reached from elevation 90.00 m down to elevation 10.00 m. The rocks in the vicinity of the upstream portal consist pebbly graywackes to pebbly mudstones. The pebbly graywackes slightly to moderately weathered or at least intersected by several wide major shear zones of highly weathered pebbly graywackes are masive. It is crossed by some major discontimuities, some of which are partly clay filled, others are unfilled.

The orientation and the traces of major discontinuities are illustrated in Figures 8.6 and 8.7. These discontinuities are favorable oriented, with respect to the slope stability, as they dip into the rock mass. In a case of two discontinuities, the line of intersection also dips into the slope. For the left channel side of the diversion tunnel upstream portal, howener, the orientation of the discontinuities is less favorable and at the elevation 41.50 to 47.30 m two small wedges were inferred (Figure 8.8). They might become unstable once the excavationce the excavation reaches the final depth, provided that the discontinuities are assumed persistent.

The geologic condition at the downstrean portal is more critical as the rocks more weathered and fractured than at the upstream side (Figures 8.8 and 8.9). The rocks of downstream portals similar to the zone above elevation 60.00 m at the upstream portal.

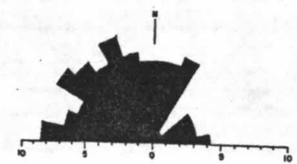
## 8.4 Results and Discussions of Rock Slope Stability Analyses

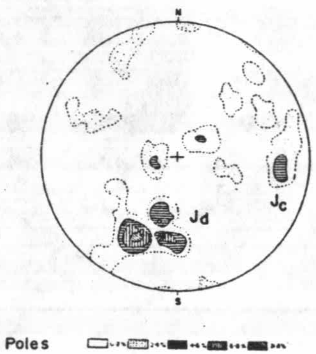
Eighteen structural regions on both portal slopes were divided and the Geomechanics Classification of jointed rock masses as well as the graphical analyses of slopes stability using Markland's (1972) and Hendron's (1971) methods were performed. The results of the analyses for 11 subareas at the inlet portal slopes (Figure 8.10) and 7 subareas at the outlet portal slopes (Figure 8.11) are discussed below.




**EXPLANATION**

| | |
|---|---|
| (symbol) | Fault or shear zones |
| 183/25° | Dip direction and angle of jointing planes |
| Gwke | Pebbly greywacke to pebbly mudstone |
| Sark | Subarkosic sandstone |




ORIGINAL GRAPHIC SCALE
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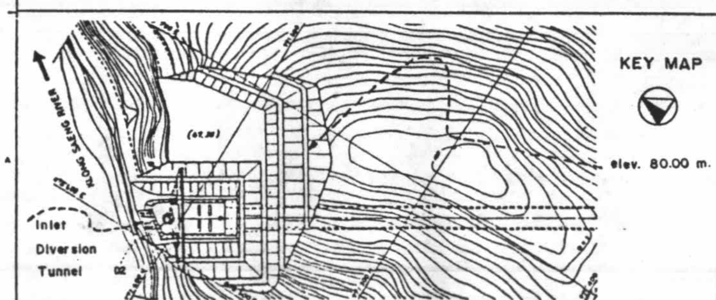


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|---|---|
| UNDER SUPERVISION OF : Dr. NOPADON MUANGNOICHAROEN, Dr. WINNART GUMPERAYARNNON | MASTER THESIS TITLE ON : GEOMECHANICAL ASPECTS AND THEIR APPLICATIONS TO THE DIVERSION TUNNEL STABILITY AT CHIEW LARN DAM SITE, CHANGWAT SURAT THANI |
| **Figure 8.6** PURSUED BY : DANUPON TONNAYOPAS | SIMPLIFIED CROSS-SECTION OF UPSTREAM PORTAL SLOPES |

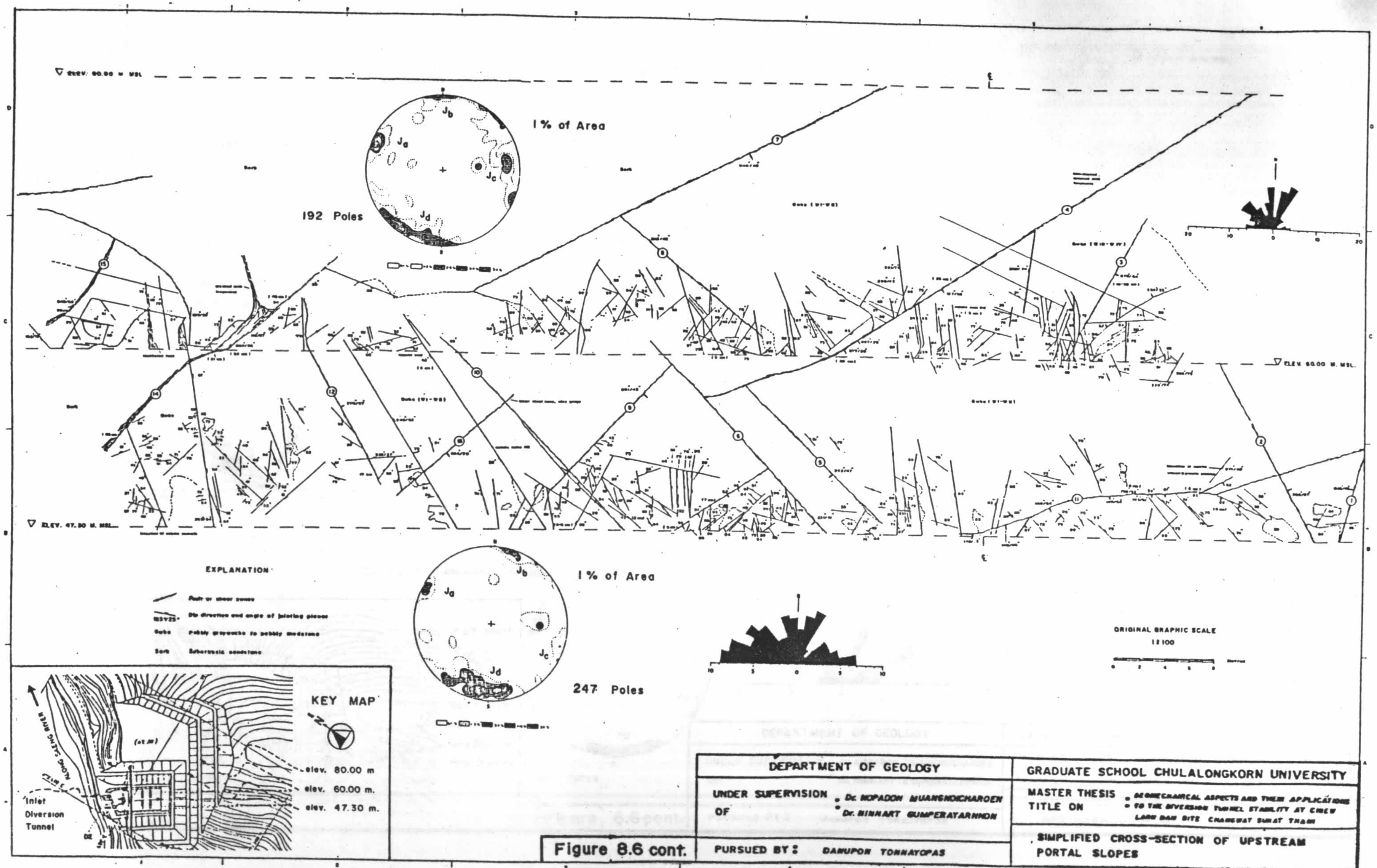



Figure 8.6 cont.

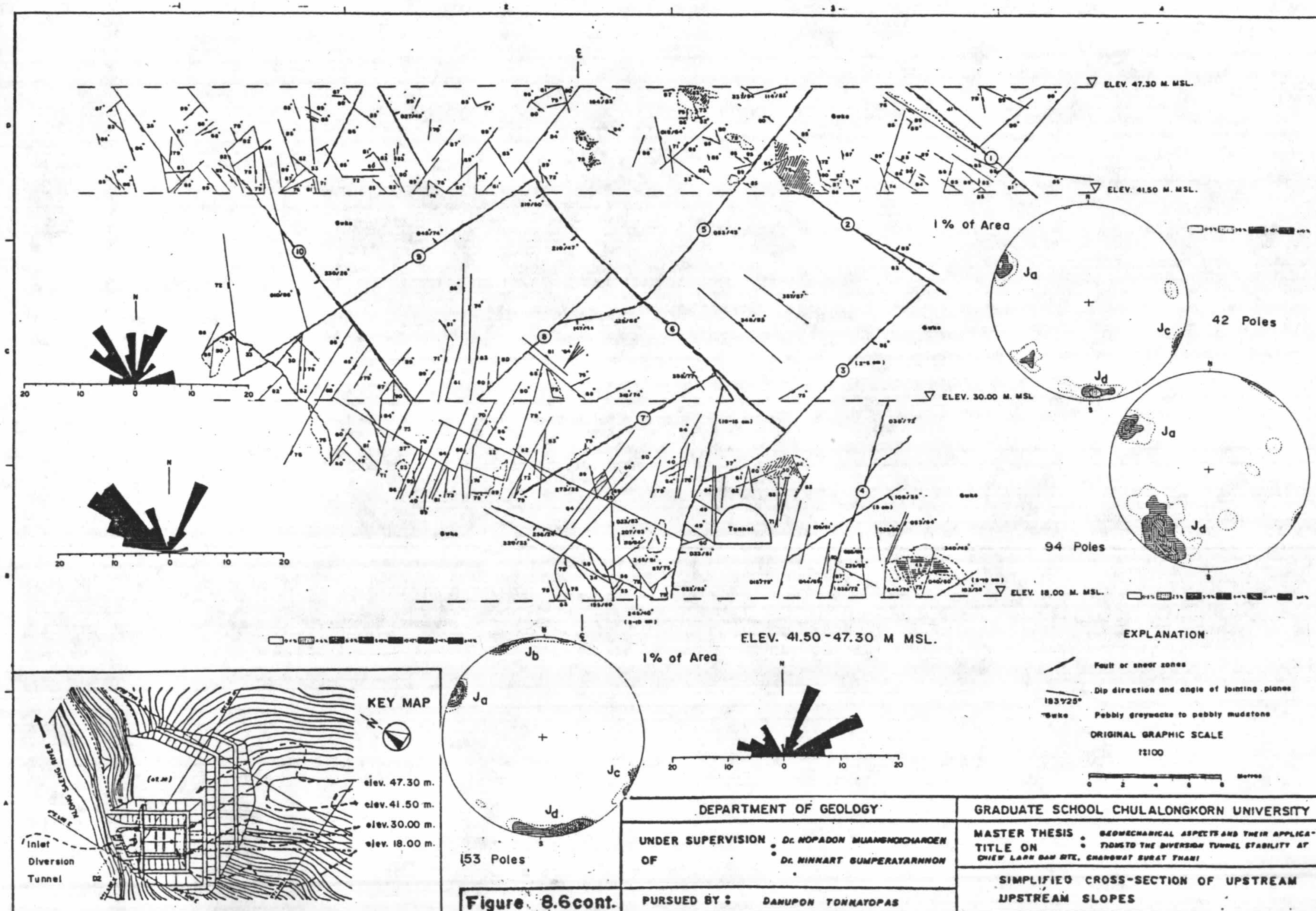
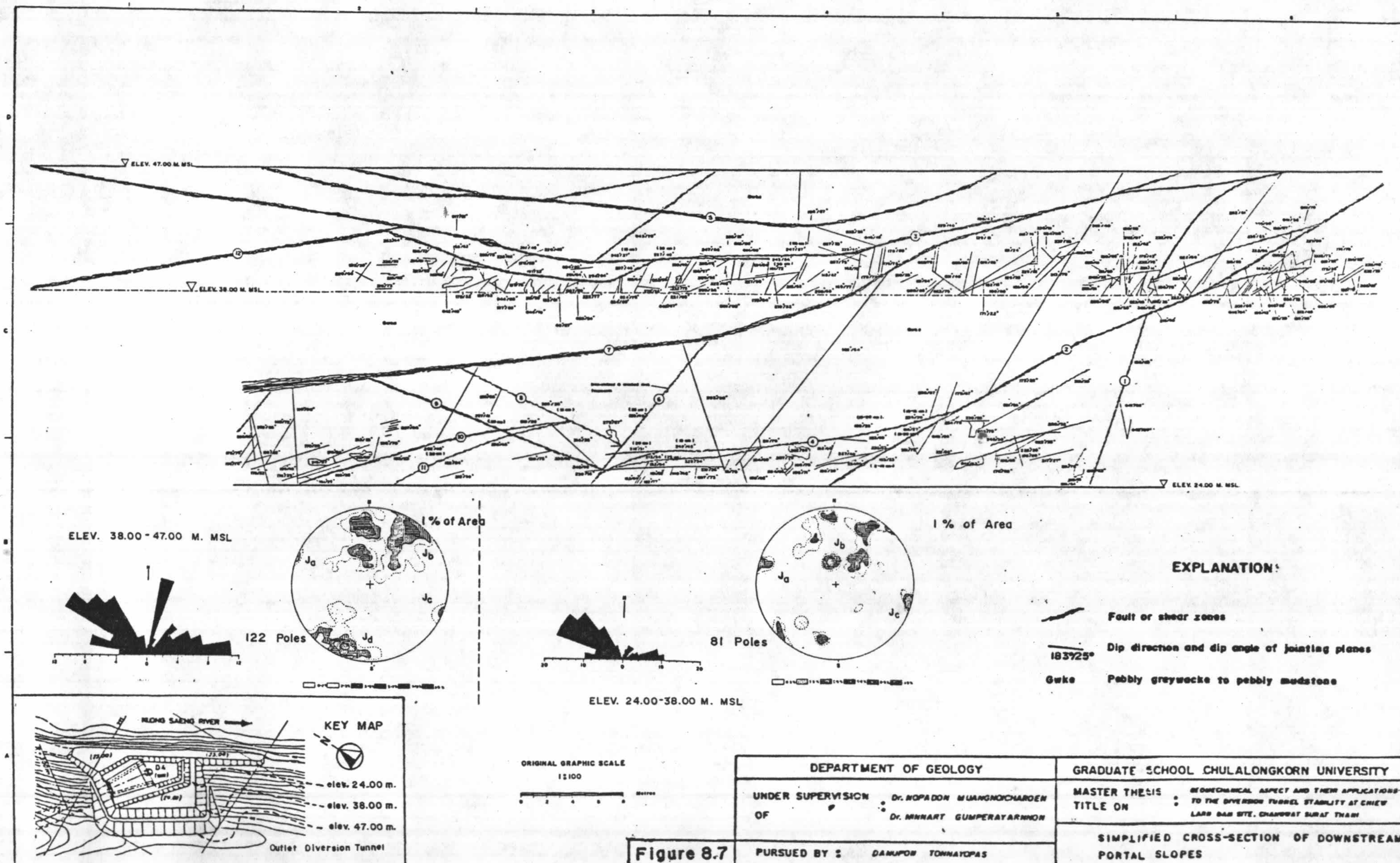


Figure 8.6cont.



ELEV. 47.00 M. MSL
ELEV. 38.00 M. MSL
ELEV. 24.00 M. MSL
ELEV. 38.00 - 47.00 M. MSL
122 Poles
1 % of Area
Ja
Jb
Jc
Jd
81 Poles
ELEV. 24.00-38.00 M. MSL
EXPLANATION:
Fault or shear zones
Dip direction and dip angle of jointing planes
183°/25°
Gwke
Pebbly greywacke to pebbly mudstone
KEY MAP
elev. 24.00 m.
elev. 38.00 m.
elev. 47.00 m.
Outlet Diversion Tunnel
ORIGINAL GRAPHIC SCALE
1:100
DEPARTMENT OF GEOLOGY
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Dr. MINNART GUMPERAYARNNON
PURSUED BY : DANUPON TONNAYOPAS
GRADUATE SCHOOL CHULALONGKORN UNIVERSITY
MASTER THESIS TITLE ON : GEOMECHANICAL ASPECT AND THEIR APPLICATIONS TO THE DIVERSION TUNNEL STABILITY AT CHIEW LARN DAM SITE, CHANGWAT SURAT THANI
SIMPLIFIED CROSS-SECTION OF DOWNSTREAM PORTAL SLOPES


Figure 8.7

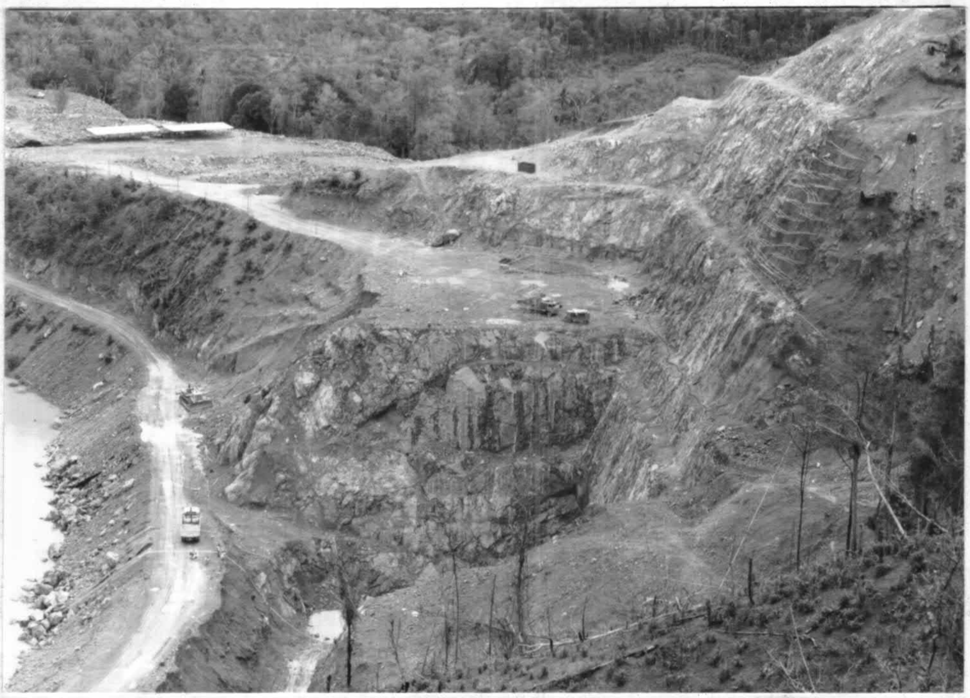

Figure 8.8 Panorama of upstream diversion tunnel.

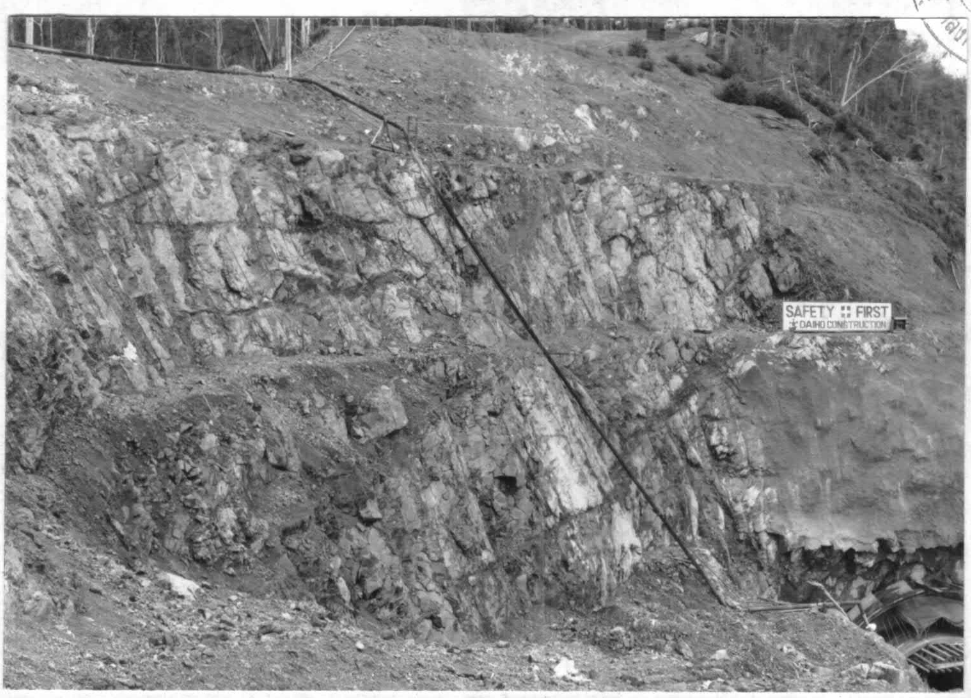


Figure 8.9 Panorama of downstream diversion tunnel.

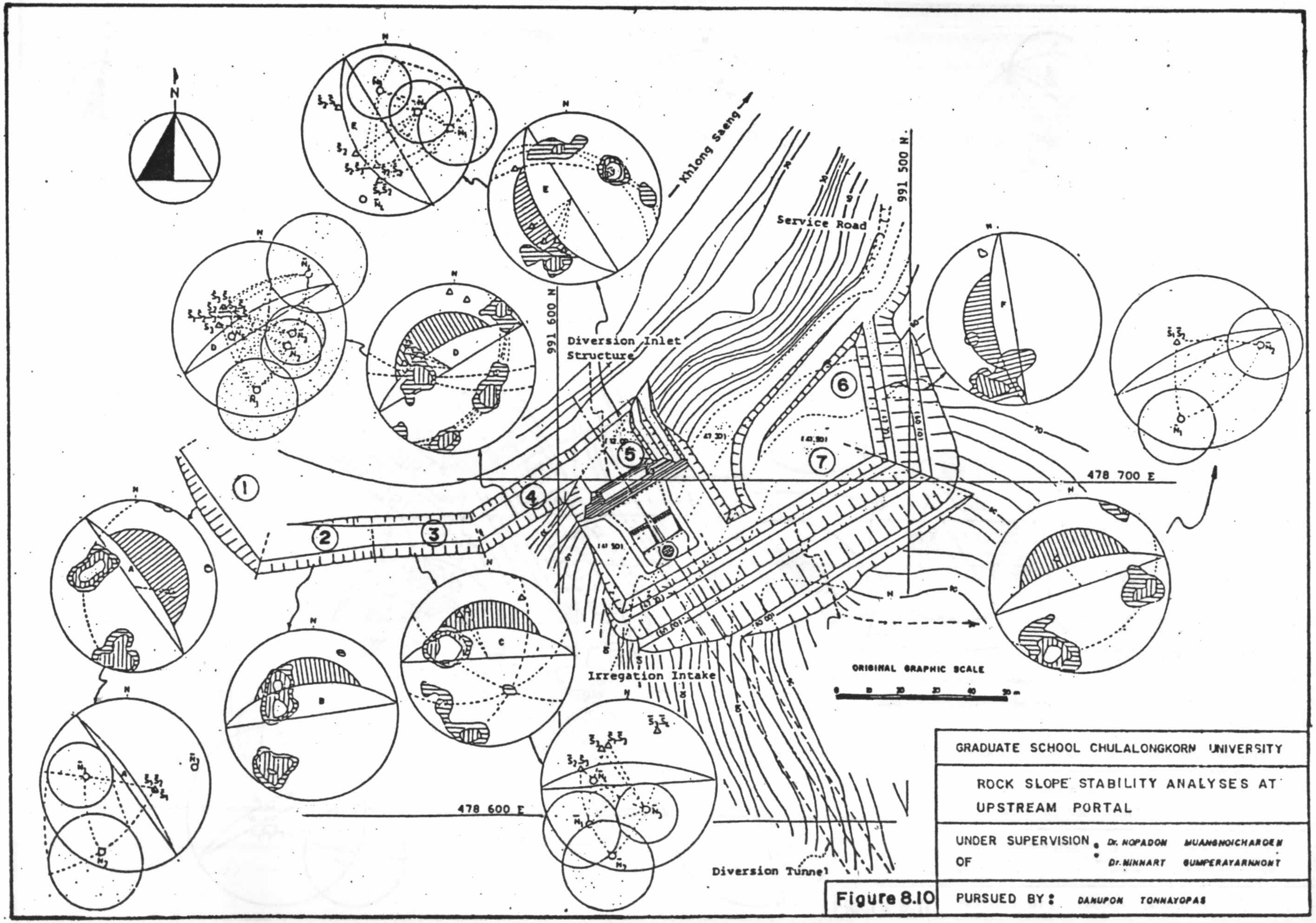
N
Khlong Saeng
Service Road
991 500 N.
991 600 N
Diversion Inlet Structure
478 700 E
478 600 E
Irregation Intake
Diversion Tunnel
ORIGINAL GRAPHIC SCALE
GRADUATE SCHOOL CHULALONGKORN UNIVERSITY
ROCK SLOPE STABILITY ANALYSES AT UPSTREAM PORTAL
UNDER SUPERVISION OF : Dr. NOPADON MUANGNOICHAROEN
Dr. WINNART SUMPERAYARNNONT
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Figure 8.10

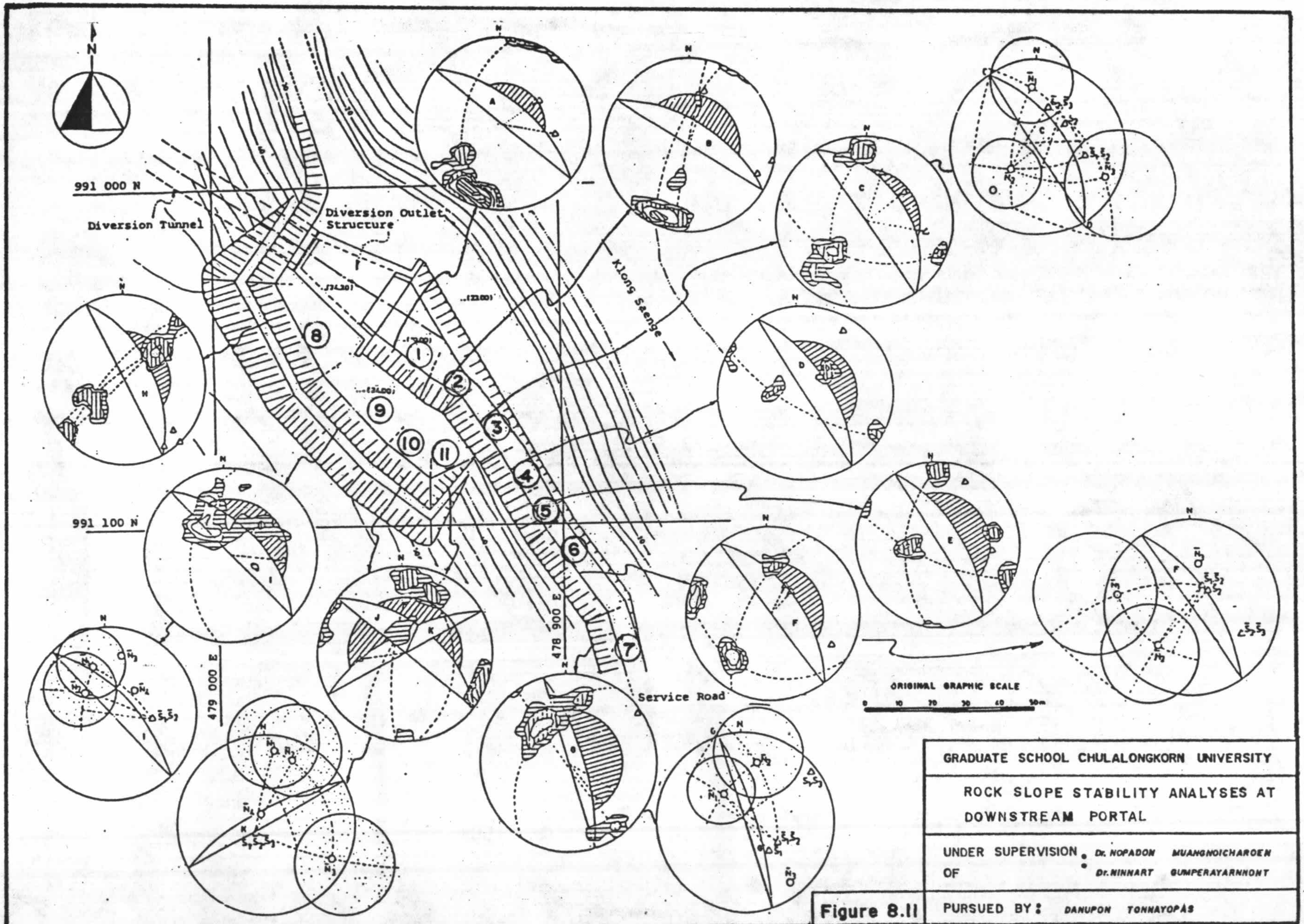
N
991 000 N
991 100 N
479 000 E
478 900 E
Diversion Tunnel
Diversion Outlet Structure
Klong Saenge
Service Road
ORIGINAL GRAPHIC SCALE
GRADUATE SCHOOL CHULALONGKORN UNIVERSITY
ROCK SLOPE STABILITY ANALYSES AT DOWNSTREAM PORTAL
UNDER SUPERVISION OF : Dr. NOPADON MUANGNOICHAROEN
Dr. WINNART GUMPERAYARNNONT
PURSUED BY : DANUPON TONNAYOPAS

Figure 8.11

### 8.4.1 Upstream Portal Slope

#### 8.4.1.1 Subarea U-1

The rocks are highly weathered pebbly graywackes with very poor quality rock masses. A set of discontinuities is generally close spacing. An occasional seepage at the hight 1-2 m above the slope toe always occurs after the rainfalls. The set of the discontinuities is of the continuous shear planes with clay gouge of about 10-15 cm thick an bands of crushed rock fragments. The joint surface coating material is commonly limonite with very low to low wall strength. The observed stability condition is unstable. The potential failure might be the rock falls and rock slides in both in both plane and wedge failure types. From the analysis, it shows that the subarea is unstable, the failure may occur in the direction of N60°-85°W with a plung of 40°- 50°. The safety factor is 0.66-1.34.

For the further excavation, the orientation of a stable slope should trend S57°W or roughly east-west with the dip wqual to or less than 44° to the north.

#### 8.4.1.2 Subarea U-2

The rocks are highly weathered pebbly graywackes with the very poor quality rock masses. The most continuous seepage during the rainy season was found at 1-2 m above the slope toe. The potential failure surface was found only in a 5 m section on the west side of slope where the small wedge failures were presented. But the stability condition from the analysis for the whole subarea is unstable. The potential failure is in the diredtion plunge of N34°W/39°and N24°W/30° with the safety factors equal to 0.95 and 1.07 respectively. The orientation of a statble slope in this subarea should be trending S87°W or roughly east-west with a dip equal to or less than 40° to the north.

8.4.1.3 Subarea U-3

The rocks are highly weathered pebbly graywackes with fair quality rock masses. A large continous seepage occurs during rainfall at about 1 m above the slope toe. The observed stability is stable and so is the stability condition from the analysis result.

8.4.1.4 Subarea U-4

The rocks are moderately to highly weathered pebbly graywackes with the fair quality rock masses. Two main potential failure surfaces which tend to give a wedge failure along the line of intersection of these planes are illustrated in Figure 8.10. The analysis result indictes that the stability condition is unstable. The direction of failures will be plunging 50°- 52° to S80°- 84° E with the safety factor equals to 0.51-0.55. The orientation of a stable slope should be trending N36 W with dip equal to or less than 60° toward the northeast.

8.4.1.5 Subarea U-5

The rocks are moderately weathered pebbly graywackes with the poor quality rock masses. The observed stability condition is unstable, the plane failure might occur on the weakness plane which moderstely dips follow that of the slope face in direction

N33°W/55°. From the analysis result, the subarea is unstable. The potential failure will occur in the direction S24°- 60° W/46°- 47°. with the factors of safety wqual to 0.75 and 0.78. The orientation of a stable should be trending S33°E with the dip equal to or less than 42° to the southwest.

8.4.1.6 Subarea U-6

The rocks are subarkosic sandstones and slightly weathered pebbly graywackes. The subarkosic sandstones overlie the pebbly graywackes with a sharp contact as illustrated in Figure 8.6. The rock mass quality is good for both rock types. The observed stability condition is stable and from the analysis result, it is also stable.

8.4.1.7 Subarea U-7

The rocks are slightly weathered pebbly graywackes with the very poor quality rock masses. Two joint sets with limonite veneers and with clay filled are mainly distributed. An occasional seepage after rainfalls is normally found. The observed stability condition is unstable. A small scale wedge failure might occur along the lines of intersection of the weak planes. From the analysis result, the stability condition is unstable, the wedge failures may occur on the lines of intersection is the direction N44°W/57° with safty factor equals to 0.81. The orientation of a stable slope should be trending S65°W with dip equal to or less than 57° to the northwest.

8.4.2 Downstream Portal Slope

8.4.2.1 Subarea D-1

The rocks are slightly weathered pebbly graywackes. There are two main sets of discontinuity. One is the discontinuous vertical joints dipping in the same direction as that of the slope face. The other is of the shear planes with the thin layers of crushed rock fragment. An occasional seepage through the shear planes is common after a heavy rainfall. The rock mass quality is good. The observed stability condition from the analysis result is also stable.

8.4.2.2 Subarea D-2

The rock are slightly to moderately weathered pebbly graywackes largely with limonite staining. Two main continuous joint sets plus few random joints are distributed. The rock mass quality is good. The observed stability condition is also good. The stability condition from the analysis result is also stable.

8.4.2.3 Subarea D-3

The rock are fresh to moderately weathered pebbly graywackes with the predominant veneers of limonite. An occasional rock slide might occur on any plane of weakness which emerges on the master joint plane, should the slope be further excavated to the joint. The stability condition from the analysis is unstable. The potential plane failure is in the direction N50° E/44° with the factor of safety equals to 0.70. The orientatio of a stable slope is trending N33°W with the dip equal to or less than 44° to the northeast.

8.4.2.4 Subarea D-4

The rocks are slightly to moderately weathered pebbly grawackes generally with limonite veneers. The rock masses quality are fair. The observed stability condition and the analysis indicate that it is stable.

8.4.2.5 Subarea D-5

The situation of stability condition and result are the same as Subarea D-4, that is, the subarea is rather stable.

8.4.2.6 Subarea D-6

The rocks are moderately pebbly greawackes in the lower part and moderately to highly weathered in the upper part. The limonite veneers are commonly occured. A large amount of seepage occured at 2.5-4.5 m above the slope toe and tended to be continuous through the rainy season. The rock masses quality are poor. The observed stability condition is unstable. The plane failure tends to occur with an aid of the toppling from the vertical tension joint. The stability analysis result indicated that it is unstable and the sliding may occur in the direction N45°E/62° and N29°E/61° with the safety factors equal to 0.51 and 0.49 respectively. On this slope a plane failure can occur on the discontinuity itself.

If the lubricating effect from groundwater as well as the water diversion is taken into a consideration, this slope face should be designed conservatively. At least the slope angle should be equal to dip angle of the potential plane failure. The orientation of a stable slope should trend N34°W with the dip equals to or less than 62° to the northeast.

8.4.2.7 Subarea D-7

The rocks are moderately weathered pebbly graywackes. There are three main predominant joint sets while the shear planes with the filling of crushed rock fragments, clay gouge and limonite are predominance. The rock masses quality are poor. The observed stability condition is unstable. The potential failure might occur on the shear plane in the direciton almost parallel to slope face. The stability condition from the analysis result is unstable with the sliding direction of S37°/44° and S46°E/42° on the shear plane. The safety factor is 0.79 for both directions of failure. The orientation of a stable slope should be trending N14°W with the dip equals to or less than 53° to the northeast.

8.4.2.8 Subarea D-8

The rocks are slightly weathered pebbly graywackes. The shear planes with crushed rock fragments, clay gouge and limonite are predominant. An occasional seepage is largely through the shear planes after a rainfall. The rock masses quality are good. The observed stability condition as well as the analysis result suggest a stable condition.

8.4.2.9 Subarea D-9

The rocks are moderately to highly weathered pebbly graywackes. The multiple shear planes are generally distributed in this subarea. The rock masses quality are poor the fair. The observed stability condition is unstable. The rock falls and small wedge failures along the intersection line of the shear planes were observed. The stability condition from the analysis is unstable. The plane failure with an aid from toppling may occur in the direction S80 E/38 with the safety factor equals to 0.80. The orientation of a stable slope should be trending N40°W with the dip equal to or less than 52° to the northeast.

8.4.2.10 Subarea D-10

The rocks are moderately to highly weathered pebbly graywackes. The rock masses quality are fair to good. The observed stability condition is good. The stability condition from the analysis is stable.

8.4.2.11 Subarea D-11

The rocks are moderately to highly weathered pebbly grawackes in the lower part and highly weathered in the upper part. The distribution of discontinuties are the same as in subarea D-10, the only difference between the two subareas is the slope face orientation. The rock masses quality are poor to fair. The observed stability condition is stable though some rock falls from the upper part might occur. The stability condition from the analysis is stable.