Soft Sedimentary Rock Slopes Design of Diversion Tunnel

Dr. Warren Wangryul Jee
(Tunnel Specialist, KICT, Ilsan, Korea
Managing Director, Deahan Consultant, Seoul, Korea
Adjunct Professor, Hanyang University, Seoul, Korea)

Abstract

Several remedial works were attempted to stabilize the collapsed area of the inlet slopes of diversion tunnel, but prevention of any further movement was being only carried out at beginning stage by filling the area with aggregates and rock debris, after several cracks had been initiated and developed around the area. The extra specially developed folding zone is consisted with highly weathered Greywacke and Black shale. The suggested solution is to improve the properties of the rock mass of failed area by choosing the optimum level of reinforcement through the increment of slope rock support design so as to control the movement of slopes during the re-excavation.

The Bakun hydroelectric project includes the construction of a hydroelectric power plant with an installed capacity of 2,520MW and a power transmission system connecting to the existing transmission networks in Sarawak and Western Malaysia. The power station will consist of a 210m height Concrete Faced Rockfill Dam. During the construction of the dam and the power facilities the Balui River has to be diverted of the tunnels is 12m and the tunnel width is 16m at the portal area. This paper describes the stability analysis and design methods for the open cut rock slopes in the inlet area of the diversion tunnels. The geotechnical parameters employed in stability calculations were given as a function of four defined Rock Mass Type (RMT) which were based on RMR system from Bieniawski. The stability calculations procedure of the rock slopes are divided into two stages.

In the first stage, it is calculated for the stability of each "global" slope without any rock support and shotcrete system. In the second stage, it is calculated for each "local" slope stability with berms and supported with rock bolts and shotcrete. The monitering instrumentation was performed continuously and some of the design modification was carried out in order to increase the safety of failed area based on the unforeseen geological risks during the open cut excavation.

1. Introduction

Located in a typical rainforest area on the island of Borneo, with mountains in the middle of Sarawak, Bakun has a huge hydroelectric potential. The strong economic development in Malaysian peninsular requires high increase in electricity supply. Dong-Ah Construction Industrial Co., Ltd., Seoul has got the contract for the river diversion works of the Bakun Hydropower Project, located in Sarawak, Malaysia. The power station will consist of a 210m height Concrete Faced Rockfill Dam(CFRD), powerhouse at six generating units, and the spillway with gated headworks. The installed capacity of the power scheme will be 2,520MW. During the construction of the dam and the power facilities the Balui River has to be detoured by three diversion tunnels with a length of some 1400m each. The excavation diameter of each tunnel is between 13m and 13.5m depending on the in-situ rock conditions. The tunnel width is widely designed 16m at the portal area.

In the diversion works, there are two rock slopes having a height up to some 75m at the outlet and some 60m at the inlet area. It is planned to excavate with a single typical geometry that is consisted of a stepped contour with 5m wide berm at every 15m of height. The inclination of all slopes is planned to about 60°. Two stages were considered for the stability calculation of the rock slopes. The stability of each "global" slope (overall inclination about 48°~50°) must be assured without any rock support and shotcrete as it would be nearly impossible to stabilize such high slope. But the "local" slopes (inclination about 58°~60°) with berms which are supported with rock bolts and shotcrete must be stable.

The stability analysis for plane wedge failure has been performed by limit equilibrium analysis. "Swedge" software was used for wedge analysis of open cut slopes. In the slope stability analysis, the influences of external load as well as earthquake loads are included. Water pressures in the failure planes and tension cracks are computed by assuming that extreme condition of every heavy rainfall may occur, and that in consequence the fissures are completely filled with water. It is assumed that the pressure varies from zero at the free faces to a maximum value at the line of intersection on the two failure planes.

2. Geological Survey

The First Indication of suitability of the hydropower dam site is the morphology of the Bakun area, which is characterized by prominent 2km wide and up to 700m high NE-SW trending mountain range clearly indicated in the topological map 1:50,000 and the aerial photographs. The Balui River cuts this range nearly at right anger i. e. in southeast-northwest direction and forms gorge-like valley with slopes up to 60°. Except for the localized quaternary deposit(eluvial, colluvial and alluvial) of 0.5m to 5m in thickness, the geology of the Bakun project area belongs to the several thousand meters thick Belaga Formation deposited from the late Cretaceous to the early Tertiary in the Northwest Borneo Geosyncline. The orientation of the strata does not very much. They strike at N55E to N70E and dip at 50SE to 70SE at the site. The sequence is in the original stratigraphic order, i. e. the older strata are covered with the younger ones. The sequence was deposited in numerous sedimentary rhythms with more or less conglomeratic sandstones at the base and shale and mudstones at the top. The series contains also several intraformational conglomerates and breccias mostly as lenses. Due to the gradual faces evolution or incomplete grain size separation, all intermediate rock type ranging from clay siltstone to conglomeratic shale are present. Most rocks are highly compacted due to orogenesis.

The geographical distribution of rock types shows a concentration of strong greywacke sandstones in the Bukit Bayong and the Bukit Bakun areas with up to 100m thick massive sequences interrupted with clay or siltstone layers. Upstream of Bukit Bayong and downstream of Bukit Bakun, no massive greywacke series were observed. As the beds are striking nearly at right angle to the river valley with steep inclination, the lithological conditions on both river banks are almost the same. The other conspicuous structural elements are lineaments cutting the arc from NW and then deviating to ESE direction.

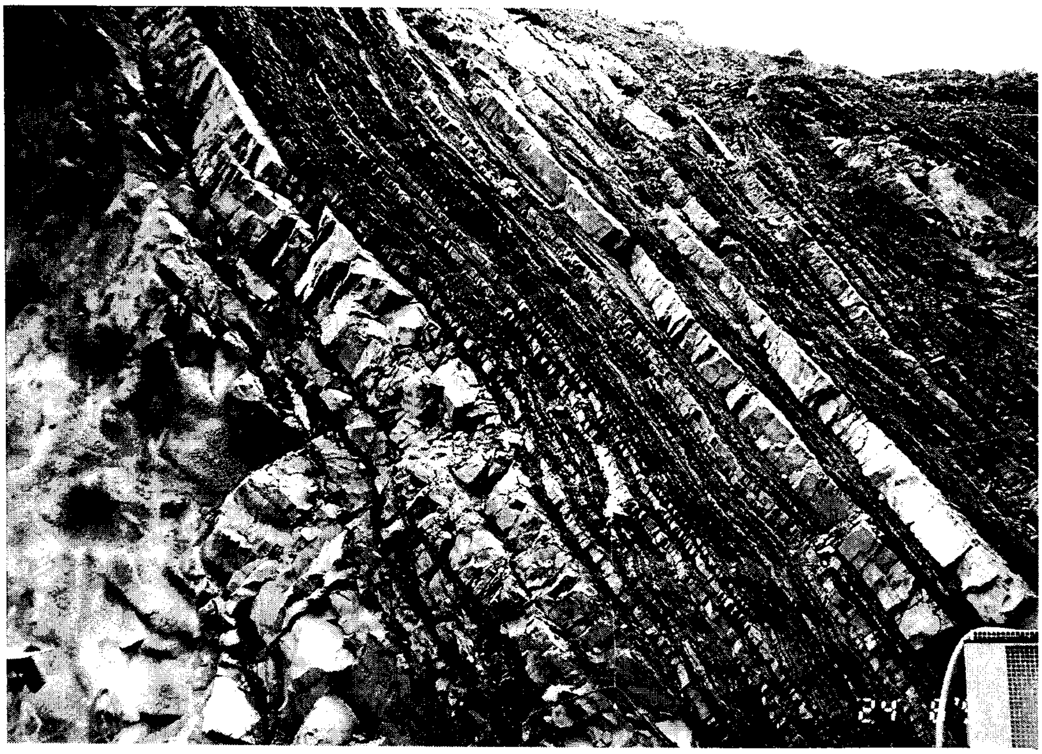


Figure 1. Geological layout of the Bakun diversion tunnelling works

The structural pattern of the geosynclinal Belaga Formation is generally steep(60° to 90°) and isoclinal. Small scale foldings of disharmonic type and overturning are frequent. The intensity of folding diminishes in the younger members, i. e. to the northwest with a simultaneous decrease in metamorphism. Slaty cleavages are mostly well developed in the argillaceous sediments and very often they obliterate the bedding completely. Faulting, i. e. shears parallel to bedding is widespread. But owing to the steep folding, the strike of faults can rarely be recognized or their throws determined. Oblique faulting occurs as well and probably some of river courses are controlled by them. At the dam site, no major fault was observed. However, several minor faults may exits at the site, as can be inferred from core drilling results.

2.1 Local geology

These sediments around the Inlet portal area have folded and faulted and occasionally intruded by filling material clays and mudstones. Near the tunnel portal, these rocks have undergone various degrees of weathering, which was found through the excavation. These laminated Bakun shales of Inlet T3 portal area has been folded into a series of synclines and anticlines, with the fold axes dipping steeply to the north and south which was found after tunnel excavation. At this T3 portal of Inlet area, this diversion tunnel passes through this chimney shaped fault zone and mixed face conditions consisting of clays and colluvium. These unique geological conditions are existing only Inlet portal area of T3 tunnel in Bakun.

3. Geotechnical Parameter

3.1 Rock types

Following groups of predominant rock types corresponding to their material properties are described. They can be summarized as follows:

- Predominant greywacke : The greywacke is mostly sandstone consisting of angular and poorly sorted quartz and feldspar grains which is estimated as an excellent aggregate for concrete mixture. Conglomerates occur only locally and consist of subrounded quartz grains and well rounded and flat shale fragments.
- Predominant shale mostly with a high percentage of silt : The siltstone as well as the fine grained sandstone consist mainly of quartz and have a silica matrix.
- Thinly interceded shale, siltstone and greywacke alternations : The shales are mostly silty. They consist of different clay minerals, quartz, calcite, and small amounts of mica. The cement is mostly calcitic.

3.2 Rock mass type and assigned parameters

The rock masses in the project area have been divided into for rock mass types(RMT). As the assignment of the different rock types to rock mass types depend mainly on the degree of weathering and the occurrence of jointing, greywacke as well as shale can occur together in the same rock mass type if their rock mass properties are similar. The rock mass type(RMT) have been listed in Table 1.

Table 1. Rock Mass Type(RMT) in the project area Bakun(Wittke, 1994 & Bakun, 1995)

RMT		Description
RMT I Very Good Rock (GW)		Thickly to very bedded fresh greywacke. The greywacke is massive of only slightly jointed. joints are generally closed and show rough surfaces. They are mostly free of clay fillings and show a low degree of separation. The spacing of the joints is greater than 1.50m. The strength of the rock mass is high and the deformability low. This rock mass type is not sensitive to water and only a little overbreak is expected during the tunneling work.
RMT II Good Rock Mass (GW&GW/SH)	a)	Fresh greywacke. The greywacke is medium to closely jointed with a moderate degree of separation.
	b)	Massive, slightly jointed shale/mudstone with bedded greywacke intercalations. The rock mass might be occasionally slightly weathered and might contain some clay fillings of slickensides. The rock mass strength will be moderate to high and the deformability moderate to low. The rock mass is not sensitive to water ; only on fillings a minor influence of water might be possible.
RMT III Fair Rock Mass (SH)	a)	Predominantly shale/mudstone with bedded shale/greywacke. The shale/greywacke interbeddings which are medium to very closely jointed, loosened or contain a moderate to high number of shear zones and possible.
	b)	Closely jointed greywacke which is moderately to highly weathered. The joints are smooth, of a high degree of separation and filled with clay. The rock mass strength will be moderate to low and the deformability moderate to high. In weathered zones the influence of water is moderate to high. Also a moderate to high overbreak is expected.

(continue)

RMT		Description
RMT Ⅳ **Poor Rock Mass (SH)**	**a)**	Very closely jointed greywacke.
	b)	Shale/mudstone with very thin greywacke intercalations and very closely to extremely closely joints. In both rock mass types the discontinuities show a nearly complete degree of separation with clay fillings, mylonitised faulted zones of frequent slickensides. The strength is low and the deformability high. This rock mass type is very sensitive to water. A high overbreak can be expected.

Most parts of the tunnels and open cut slopes were estimated to belong to RMT Ⅰ to Ⅲ while RMT Ⅳ was less present. However, actual geological condition was different from the design stages. Performed stability calculations for the open cut slopes have shown that there is no difference in the required rock support between RMT Ⅰ and RMT Ⅱ as investigated wedges were stable without any rock support. Therefore only spot bolting is necessary and RMT Ⅰ and Ⅱ have been brought together in the excavation drawings as well as in the calculation. Following nomenclature(Table 2) is used for the open cut excavations:

Table 2. RMT and related rocks

Related rocks	RMT
Fresh Greywacke	RMT Ⅰ&Ⅱ
Fresh Shale and Mudstone	RMT Ⅲ
Weathered Greywacke or Shale/Mudstone	RMT Ⅳ

In Table 3, the geotechnical parameters in dependence on the RMT which are used in stability calculations are listed. After shearing, the rock mass is able to bear shear forces up to their residual values as given in Table 3. It is measured that except for RMT Ⅰ, the residual friction angles will be the same as the peak friction angles whereas the residual cohesion will be reduced as follows:

- RMT Ⅰ&Ⅱ : residual cohesion is 1/3 of peak cohesion
- RMT Ⅲ : residual cohesion is 1/6 of peak cohesion
- RMT Ⅳ : residual cohesion is 0.1 MPa

Table 3. Parameters and safety factors for the river diversion works(Bakun, 1995)

Parameter	Unit	Rock Mass Type				Safety Factor
		I	II	III	IV	
Rock:						
Density	t/m^3	2.60	2.60	2.60	2.60	1.0
Uniaxial Compressive Strength(UCS_{Rock})	MPa	200	100	60	35	1.0
Tensile Strength(TC_{Rock})	MPa	10.0	5.0	3.0	1.8	1.0
Poisson's Ratio(ν_{Rock})	-	0.20	0.2	0.25	0.30	1.0
Rock Mass:						
Rating RMR	-	80	65	50	35	-
Deformation Mod.(E_D)	MPa	12000	8000	6000	2500	1.0
Peak Cohesion	MPa	8.0	3.5	2.0	0.7	2.0
Residual Cohesion	MPa	2.67	1.2	0.35	0.1	2.0
Peak Friction Angle	°	45	40	35	25	1.1
Residual Friction Angle	°	40	40	35	25	1.1

3.3 Discontinuities and assigned parameters

Three sets of discontinuities characterize the geological conditions of the project area

- J1 bedding plane as the dominant and most important plane of separation. In the massive greywacke it shows a low degree of separation. here, the bedding planes are mostly closed and planar.
- J2(i) and J2(ii) as the main set of transverse joints. J2(ii) is the less developed complementary set of transverse joints J2(i).
- J3 a further joint set which is directed more or less perpendicular to bedding plane. J3 of greywacke is predominantly rough and planar.

In Table 4, the orientation of the discontinuities for the right river sides shown. The values were used for the wedge analyses of open cut slopes.

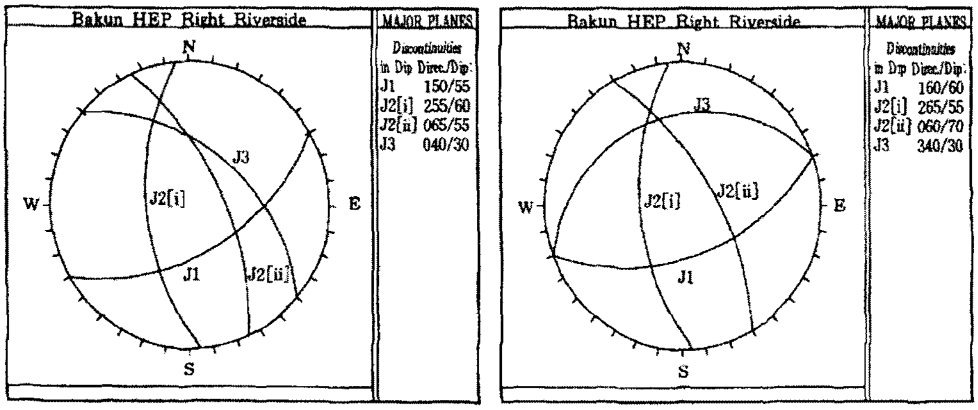


Figure 2. Discontinuities of Bakun site(Jee, 1997)

Table 4. Dip directions and dip (DD/D) of major discontinuities.

Discontinuity	Right River Side DD/D
J1	150 / 55
J2(i)	265 / 60
J2(ii)	065 / 55
J3	040 / 30

Table 5. Shear parameters on discontinuities(Bakun, 1995)

Rock Mass Type	Shear Condition	J1(Bedding)		J2		J3	
		ϕ [°]	cohes. [MPa]	ϕ [°]	cohes. [MPa]	phi [°]	cohes. [MPa]
I	Peak	40	0.60	43	0.55	40	0.60
II		35	0.40	38	0.36	35	0.40
III		30	0.25	32	0.22	30	0.25
IV		20	0.05	20	0.05	20	0.05
I	Residual	35	0.20	38	0.18	35	0.20
II		35	0.13	38	0.11	35	0.13
III		30	0.05	32	0.04	30	0.05
IV		20	0.00	20	0.00	20	0.00

The persistence of all discontinuities is set to 100% for all discontinuities. This is a conservative assumption which includes additional inherent safety as the shear strength of rock bridges is usually much greater than those of the discontinuity planes. The peak and residual shear parameters ϕ and c applied in all open cut wedge stability analysis are shown in Table 5. It has been suggested that safety factors $\eta_{\tan\phi} = 1.1$ and $\eta_c = 2.0$ will be used in main stability calculations.

4. Sedimentary Rock Slope Design of the Open-Cut Excavation

The rock slopes of the diversion work have height up to 75m at the outlet and some 60m at the inlet area. The geometry of all slopes is similar and independent of the discontinuity system. The location of the slopes is generally favorable with respect to the local discontinuity system as dipping of discontinuities is steep compared with the inclination of the local slopes. For stability calculations Hoek and Bray(1980, 1981 & 1995) suggested to make a distinction between "local" slopes 15m in height with 5m width berms and slope inclination up to 60°, and "global" slopes with a single typical geometry (Figure 3). The geometry consist of stepped contours with 5m wide berms every 15m of height.

The inclination of all slope is planned to about 60°. The stability calculations of the rock slopes are divided into two stages:

- The stability of each "global" slope (overall inclination about 48°~50°) must be assured without rock support and,
- The "local" slopes (inclination about 58°~60°) with berms which are supported with rock bolts and shotcrete must be stable.

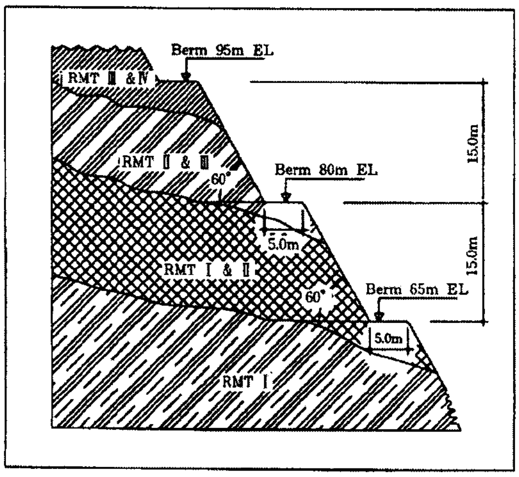


Figure 3. Typical geometry of the planned rock slopes(Jee, 1997)

4.1 Global slope stability

For the stability calculation of global wedges the inclination of sliding planes will be determined as suggested by Hoek and Bray(1981). For the Bakun Project the angle ψ_f was assumed to 50° and ψ_p was determined as follows: $\psi_p = 0.5 \cdot (\psi_f + \phi_T) \cdot f$ (1)

where, ψ_p : Inclination of the sliding plane

ψ_f : Inclination of the slope surface

ϕ_{RMT} : Friction angle within the sliding surface depended on the RMT

f : Variation of the sliding plane, here estimated to $f = 1.0$ to 2/3

It is not necessary to consider tension cracks at the surface for the global stability. Dip of all discontinuities is, apart from J3, steeper than the inclination of the global slopes so that sliding on discontinuities can normally be neglected in the estimation of global stability. The sliding angle to be investigated is flatter than dip of each discontinuity. Therefore a sliding of "global" wedges can only occur within the rock mass. The ψ_p depends on an "average friction" angle ϕ_{RMT}. This friction angle will be the average of the rock mass friction angle ϕ_{RM} and the friction angle ϕ_{J1} of that joint set nearest to the sliding plane. As the friction angles of the discontinuities are equal for RMT Ⅳ every joint set can be chosen for the stability calculation. Then, if the stability for RMT Ⅳ is given, further investigations for the better rock mass types are not necessary. According to the values for the shear parameters given in Tables 2 and 5, ϕ_{RMT} can be calculated for RMT Ⅳ to

$$\phi_{RMT,IV} = \frac{1}{2} atan(0.47+0.36) = 19.8 \approx 20^{\circ}$$

For all open cut excavation safety factors $\eta_{\tan\phi}/\eta_c = 1.1/2.0$ will be used. This leads for the friction angle $\phi_{RMT,IV,cal}$ to be applied in calculations to

$$\phi_{\mathrm{T,IV,cal}} = atan(\frac{\tan(\phi_{\mathrm{T,IV}})}{\eta_{\tan\phi}} = 18^{\circ}$$

The sliding angle ψ_p can be determined for $f = 1.0$ to

$$\psi_p = \frac{1}{2} \cdot (50+20) = 35^{\circ} \text{ and for } f = \frac{2}{3} \text{ to } \psi_p = \frac{1}{3} \cdot (50+20) = 23.3^{\circ}$$

The rock mass cohesion will be considered in the same way. Similarly, CRMT can be calculated as followed $C_{RMT} = 0.5 \cdot (C_{,\mathrm{RM}} + C_{J1,\mathrm{T}})$.

For rock mass type Ⅳ, $C_{RMT,IV} = 0.5 \cdot (0.7+0.05) = 0.375MPa$ and the cohesion to be used in the stability calculations is $C_{RMT,IV,cal} = C_{RMT,IV}/\eta_c = 0.375/2 = 0.19MPa$

$$S.F. = \frac{C \cdot A + ((R+E_v) \cdot \cos\psi_p - (W_H + E_H) \cdot \sin\psi_p + T \cdot \cos\theta - U) \cdot \tan\phi}{(R+E_v) \cdot \sin\psi_p + (W_H + E_H) \cdot \cos\psi_p - T \cdot \sin\theta}$$

where, A : area of sliding plane,

R : weight of sliding wedge,

Ev : vertical external forces,

WH : horizontal water pressure behind tension crack,

EH : horizontal external forces,

θ : angle of anchor loading against the normal of the sliding plane, positive upwards,

T : anchor forces,

ϕ : friction angle,

ψ_p : angle of sliding plane

Table 6. Input data for slope design

Input parameter	Data	Input parameter	Data
weight of rock mass	0.026 [MN/m^3]	traffic load on berm	0.030 [MN/m^3]
horizontal earthquake factor	0.10 [-]	vertical earthquake factor	0.05 [-]
cohesion on sliding plane	0.19 [MPa]	friction angle on sliding plane	18 [°]
slope inclination	50[°]	slope height	60 [m]
sliding plane inclination	23 & 35 [35°]		

Applying the following input data in to Eq.(2) leads to the safety factors
S.F. = 1.42 for a sliding plane inclination of $\psi_p = 23^\circ$ and,
S.F. = 1.43 for a sliding plane inclination of $\psi_p = 35^\circ$ and without any support. For the calculation of the global slopes, no water pressure has been taken into account due to the foreseen dense drainage system. Since a sufficient safety was given, the calculated safety factors will be greater and better than an assumed rock mass quality(RMT) due to the increasing rock mass cohesion which is the main factor for the restraining forces. As the 'global' wedges are of a reasonable height, it is valid to take the full cohesion into account for the stability calculations.

4.2 Local wedges

Stability calculation for plane wedges and wedges on two discontinuities have been performed to show the local stability of the slopes.
Following general conditions defined by Hoek and Bray (1981) must be given for a plane failure :

- dip of the failure plane must be flatter than dipping of the slope face, $\psi_f > \psi_p$
- dip of the failure plane must be greater than the friction angle on this plane, $\psi_p > \phi$
- the plane on which sliding occurs must strike in a generally parallel direction (within approximately ± 20) to the slope face and,
- lateral surfaces which provide negligible resistance against sliding must be present in the rock mass to define the lateral boundaries of the slide.

In the other cases it is more probable that wedge failure on two sliding planes may occur. For the local stability of the slopes

- a variation of different slope heights of 15m and additional 5m and 10m.
- a differentiation of plane and wedge failure calculation depending on the location of the slope related to the discontinuities.

- potential tension cracks with an inset distance of 1m near the berm, water filled or non water fined with a water level distribution : the consideration of flow nets is not necessary as the foreseen systematic drainage system in the rock mass and on the surface of the slopes will prevent water pressure forces in the possible sliding plane. The water pressure in the tension cracks can be reduced by 50% if a systematic drainage system is planned.
- the consideration of earthquake loads with, K_h = 0.10g and K_v = 0.05g and,
- the consideration of external loads or forces such as trucks on the berms. Here 60 ton trucks have been considered in the analyses which result in an additional surface pressure of 0.035MPa.

5. Results of Slope Stability Analysis

In Figure 4 the location of slopes at the inlet area is shown. The major slope directions which cover the whole range of slope directions have been taken into account for the wedge stability analysis, as shown in Table 7 and the assumed failure modes-plane or wedge are also listed. The slopes of Inlet A and B may slide on the discontinuities J1 and J2(i).

Table 7. Investigated slopes above 65m asl. of inlet and outlet area

Slope	Orientation(DD/D)	Sliding on	Failure Mode	Remarks
Inlet A	150/60	J1	plane	-
Inlet B	226/60	J1/J2i	wedge	equal outlet 5
Inlet C	046/60	J3i	plane	-
Outlet A	185/60	J1/J2i	wedge	equal outlet 5
Outlet B	211/60	J1/J2i	wedge	-
Outlet C	247/60	J2i	plane	-

Although the Inlet and Outlet area is located in greywacke of RMT II, all three rock mass types with its shear parameters have been applied in the wedge stability analysis. The stability calculation for the plane failure is base on Eq.2, while the calculations for wedge failure was performed with program 'Swedge' (Hoek, 1994). The results are summarized in Table 8.

As the results of the calculations, a provisional rock support has been determined depending only on the rock mass type and yet not on the slope direction. The support for all slopes is listed in Table 9 and shown in Figure 5.

Table 8. Stability calculation for wedge inlet 3

Rock Mass Type	I &II			III			IV		
Slope Direction (DD/D)	046/60			046/60			046/60		
Friction angle (°)	32.5			27.7			18.3		
Cohesion (MPa)	0.20			0.13			0.025		
Cohesion considered (%)	100	50	25	100	50	0	200	200	0
Applied cohesion (MPa)	0.20	0.10	0.05	0.13	0.07	0	0.05	0.05	0
Wedge Height (m)	15	10	5	15	10	5	15	10	5
Trace Length (m)	5	3	1	5	3	1	5	3	1
Calculated Wedge (t/m)	224	95	20	224	95	20	224	95	20
Wedge Geometry	Plane			Plane			Plane		
Sliding on Plane	J1			J1			J1		
Sliding Direction (DD/D)	046/30			046/30			046/30		
Wedge Face Area (m^2)	17	12	6	17	12	6	17	12	6
Rock Bolt Grid (m^2)	SB	SB	SB	16	16	9	6	6	6
Calc. Safety Factor	2.73	2.18	2.09	2.12	1.93	1.53	1.22	1.89	2.87
Shotcrete Thickness (m)	-			0.10			0.10		
Number of Wire Mesh	-			1			1		

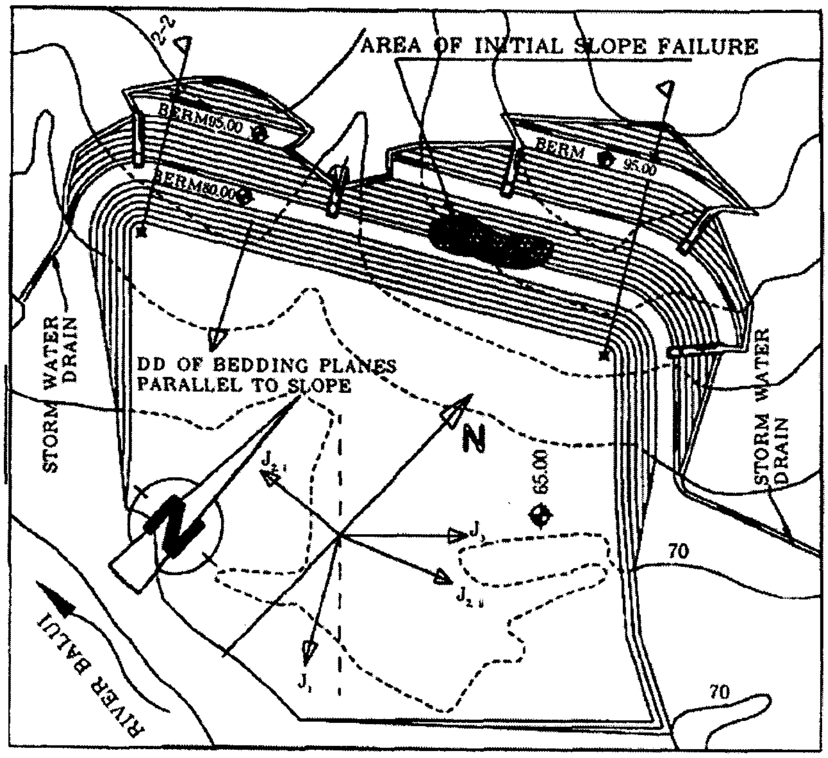


Figure 4. The location of the local investigated slopes of the inlet area

Table 9. Rock support of slopes above 65m asl of inlet and outlet area

Rock Mass Type	I & II	III	IV
Bolt Rattern (m^2)	SB	9	6
Bolt Length (m)	6 & 9	6 & 9	6 & 9
Bolt Capacity (kN)	150	150	150
Shotcrete Thickness (mm)	-	100	100
No. of Wire mesh	-	1	1
Drainage Pattern (m^2)	20	20 & 9	20 & 9
Drainage Length (m)	12	12 & 1+3	12 & 1+3

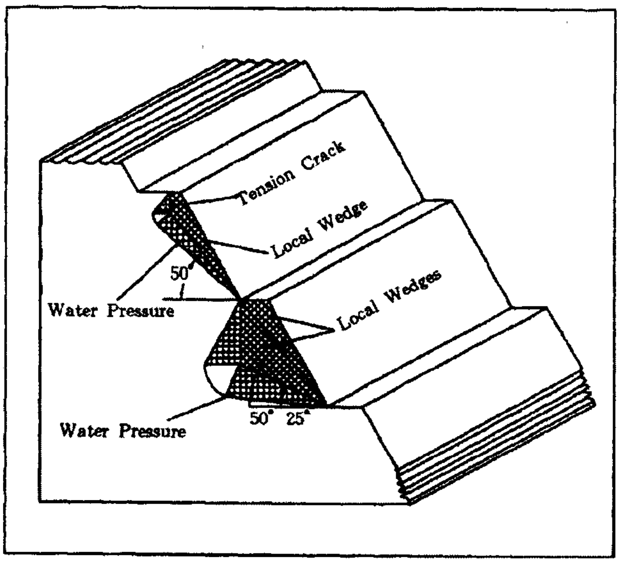


Figure 5. Plane wedge stability for local wedges

6. Inlet Slope Failure and Design Modification

A sliding failure of the inlet slope has occurred along section 3-3 for two days with consequential stages as below.

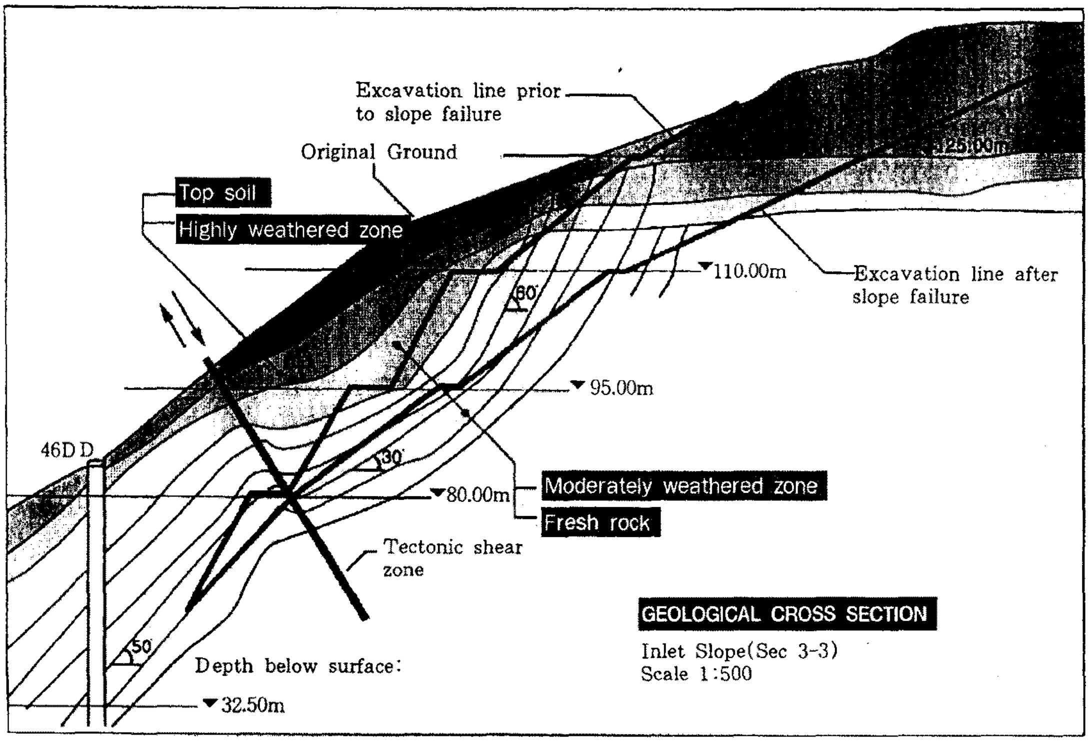
- An initial local slide on 7 July, 1996 at elevation 80~85 m asl.
- A massive huge slide up to 125 m asl which was accelerated by the heavy rainfall on 8 July, 1996.

Failure mode of the slope shows the plane failure and the failure plane develops along the J1 bedding plane. However, the initial failure is the cause of the unforeseen geological risks. The geological structure of Bakun site features a very uniform dip angles that are in the range between 55° and 60°. Unfortunately, dip angle of the inlet slope area change to flat which could not be realized during excavation of the slope(figure 6).

Actually, it is not possible to estimate for a design modification with a small area's difference during the slope excavation too. In addition, most of the monitoring data were covered at certain stable conditions, which was another confusion to the designer (Figures 7, 8). The initially failed local slope was the result of insufficient supporting dead weight because of the unpredicted flatten bedding planes(30° : dip angle) and was strongly affected by shear zone of fault at 80m asl. Misfortune never comes alone : a heavy rainfall next day accelerated the total plane slope failures up to 125m asl. Particularly, heavy rainfalls in a short periods should be the major reason of the increment of pore pressure as an additional force to reduce the shear strength of the sedimentary rocks pushes the bedding plane downward. Failure mechanism of this Inlet slope is based on the unexpected geological risks.

It should be made clear to check the rock mass rate through the slope excavation to apply the suitable support system. Contractors reported the rock excavation working progress daily and also performed the geological mapping. And rock mass rate classification was always cross checked by the client's geotechnical engineer and employer's representative at Bakun site(Jee, 1996 & 1997)

Figure 6. Geological cross section of inlet slope section 3-3



7. Conclusion

It is planned to excavate all rock slopes with a single typical geometry, as shown in Figure 3. The geometry consists of a stepped to contour with 5m wide berms in every 15m of height. The inclination of all slopes is planned to about 60°. However, on 7 July, 1996 sliding of section 3-3 of Inlet slope has started, the sliding has occurred along J1 bedding with DD/D=150°/35°, nearly parallel to the dip direction. The main reason for the slope failure was the unexpected flat inclination of the bedding plane D=35° which has been excavated too steep compared with original design. Therefore, modified inclination of the excavated slopes should follow the natural dip angle of the rock mass. Instead, the systematic rock support will be replaced with spot bolting and shotcrete below 80m asl and without any support above 80 asl.

As a result of the quantity of soil and rock excavation increases while the rock support reduces. Consequently, the slope will be stable as no sliding can occur theoretically. However, it can happen that during excavation undercutting of shale or greywacke layers occurs so that sliding is possible. The slope failure was caused by a sudden and unforeseen change of the geological conditions.

In order to ensure the future slope stability, required design modifications were studied and the above rehabilitation strategy was finally enforced.

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