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EVALUATION OF THE DAM GEOLOGY AND GEOLOGICAL RISK AT THE NAMNGIEP 1 HYDROPOWER PROJECT*

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SUMMARY

The surface of the mountain at the dam site is covered by talus and a stratified structure below has alternating layers of sandstone and mudstone having rock class of CM to CH. On this has been developed the dam site. As general the bedding planes on the both abutment are moderately dipping with 8° in downstream direction and 8° in riverbed direction. And around the middle of the dam axis the dip angle of the bedding plane is 15° to 25° in the same direction. On the right abutment the fold axis is continuous in the upstream and downstream directions and geological formation shows a steep profile, but it becomes moderate again at the higher elevation. No outstanding fault has been observed and a box fold has been formed at the steep slope area. Flexural-slip associated with fold formation has developed around the fold axis. It is extrapolated that flexural-slip occurred before solidification based on the observation that blocks formed by joint

* *Évaluation de la géologie du barrage et du risque géologique sur l'aménagement hydroélectrique de Namngiep 1*

sets intersecting with the bedding planes are observed in the sandstone layer and ductile deformation are frequently observed in the mudstone layer.

Simultaneously definite striation of reverse-fault sense harmonized well with flexural-slip observed in some fine sandstone indicates that coarse sandstone and fine mudstone had flexural-slip developed in semi-brittle and ductile condition in geological time respectively. The weak layers are 8 in number and are confirmed on the bottom of the dam and both the abutments, and they are issues to consider specific strength and continuity of weak layers.

Physical properties of the rock were determined based on observation of outcrops and drilling cores, in-situ block shear tests and laboratory tests. Physical properties of rock mass were evaluated based on Hoek-Brown failure criterion for each part of the foundation rock. Physical properties of weak layers were determined based on the shear box tests of the disturbed samples, liquid limit tests and plastic limit tests. The total strength of the weak layer can be estimated from the component of weak layers. In addition, X-ray diffraction analysis was conducted in order to verify that these fine particle fractions in the weak layers do not include any swelling clay (for example, Smectite) which might significantly degrade physical properties. Besides detailed observation by means of CT scanning was conducted in order to examine the continuity of a fractured part.

Geological risk should be evaluated adequately for the dam construction in the BOT scheme project. The risk to a hydropower project is high compared with other infrastructure projects because large amount of project cost depends on geological condition. Therefore, adequate evaluation of geological risk and diversification of geological risk are very important factors. In addition, highly accurate geological data helps adequate evaluation of geological risk. Drilling technology especially is one of the most important factors to support geological evaluation.

In this paper, the methods to evaluate the dam foundation rocks are discussed in the feasible study phase to the execution phase. Evaluation of the geological risk and the method to reduce the geological risk are discussed based on actual geological data.

Keywords: Geology, Geological Investigation, Gravity Dam, Nam Ngiep 1 Hydropower Project.

RÉSUMÉ

Le barrage a été construit sur une couche de surface recouverte de cônes d'éboulis, et constituée de grès et d'argilite de classe CM à CH. Les couches des deux appuis sont inclinées modérément de 8° vers l'aval, et 8° vers le lit de

la rivière. Pour cette raison, au total, 8 couches faibles ont été identifiées dans la zone immédiatement en dessous du barrage et dans les deux culées. Leur solidité et leur continuité demeurent des problèmes à résoudre.

L'évaluation de la roche de fondation du barrage a été basée sur l'observation des surfaces de sol excavées, l'observation des noyaux de forage, les excavations de galerie d'accès de test, les essais de cisaillement in situ et les tests de roche en laboratoire pour leurs caractéristiques techniques. Quant au remplissage, les propriétés de résistance des couches faibles ont été évaluées par les tests de cisaillement direct, les essais de compression triaxiaux et les tests de limite de plastique à l'aide d'échantillons perturbés/non perturbés. En particulier, des tests quantitatifs utilisant l'analyse en laboratoire, y compris l'analyse d'image CT, l'analyse des rayons X et l'analyse de rayons X fluorescents ont été effectués pour confirmer l'existence de la fraction à grain fin dans les structures écrasées et pour identifier leurs matériaux.

Lors de la construction d'un barrage, il est nécessaire d'évaluer de manière appropriée, en plus de l'évaluation de la fondation du barrage, les risques géologiques dans le cas du projet utilisant le schéma BOT hydroélectrique. Alors que les risques géologiques sont des risques associés à tous les ouvrages, y compris les projets de construction de routes, les impacts sont particulièrement importants dans un BOT hydroélectrique dont la structure souterraine constitue une proportion importante. Ainsi, l'évaluation, la dispersion et la réduction appropriées des risques géologiques sont les conditions pour diriger le projet vers le succès. En outre, chaque test nécessaire pour l'évaluation est considérées comme un facteur important, tout comme la compétence technologique associée à l'échantillonnage du noyau de forage, et la réduction des incertitudes avec l'utilisation de données très précises peut être considérée comme l'une des conditions pour également mener le projet à la réussite.

Dans ce document, les détails de l'étude géologique lors des étapes d'évaluation de faisabilité, de préparation et de mise en œuvre, qui complètent en l'évaluation de la fondation rocheuse du barrage sont décrits pour débattre de l'évaluation, des mesures de réduction et des résultats des risques géologiques basés sur des données réelles.

Mots-clés: Geologie, Essai Sur Modele Hydraulique, Barrage-Poids, Projet D'energie Hydraulique Nam Ngiep 1.

1. INTRODUCTION

The Nam Ngiep 1 (NNP1) hydropower project (the Project) is located along the Nam Ngiep River, which is a tributary of the Mekong River, 145 km northeast of Vientiane, the capital of Lao PDR and 50 km north of Paksan city.

The Project planned on a Build-Operate-Transfer (BOT) basis and as an independent power producer (IPP) project.

The Project consists of a main dam and a re-regulation dam. The crest length and dam height of the main dam, Roller Compacted Concrete (RCC) gravity dam, are 535.5 m and 167.0 m, respectively. The reservoir created by the main dam will store around 2 billion m^3 of water with an effective storage capacity of around 1 billion m^3 for generating electricity of the maximum output of 272.0 MW that will be exported to Thailand. The re-regulation dam is a conventional concrete gravity dam with labyrinth spillway, The major construction works commenced in October 2014 and has been approximately 80 % complete as of the end of August 2017. The aim of the Project is to commence commercial operation in January 2019.

In this paper, the dam geology and geological risk at the Project have been evaluated as follows;

1) Geological investigation, geological evaluation of dam foundation, weak layer and characteristic in the NNP1
2) Geological risk and risk reduction measures and results

2. GEOLOGICAL INVESTIGATION AND EVALUATITON

2.1. History of Geological Investigation

The history of geological investigation is divided into three phases being the Feasibility Study (FS) phase, Detailed Design (DD) phase and Construction (Co) phase as shown in Table 1.

Table 1
Summary of Geological Investigation

ITEM / PHASE	FEASIBILITY STUDY	DETAILED DESIGN	CONSTRUCTION
1. Evaluation of foundation rock			
Observation of dam foundation	only survey	only survey	Done
Observation of drilling core	5 holes (600 m)	42 holes (3,240 m)	26 holes (812 m)
Seismic prospecting	-	5 line (2,500 m)	-
Observation of test adits	-	2 Nos (198.8 m)	-
In-situ block shear test	-	12 specimens	-
Borehole scanner	-	-	22 Nos (792 m)
Physical and Mechanical properties of fresh rock	-	56 specimens	32 specimens

ITEM / PHASE	FEASIBILITY STUDY	DETAILED DESIGN	CONSTRUCTION
2. Evaluation of weak layer			
Shear box test	-	-	42 specimens
Tri-axial compression test	-	-	24 specimens
Bulk density of soils and rocks	-	-	50/32 specimens
Density test for soil particle	-	-	50 specimens
Liquid/plastic limit of soils	-	-	52 specimens
Particle size distribution	-	-	31 specimens
Compression test for rocks	-	-	30 specimens
CT scanning	-	-	14 specimens
X-ray diffraction (XRD) analysis	-	-	58 specimens
X-ray fluorescence analysis			12 specimens

The Japan International Cooperation Agency (JICA) conducted geological investigation at the initial stage of the FS phase in 2002 as requested by the Lao Government in order to study the location of the dam axis and dam type.

The Kansai Electric Power Company (KANSAI) carried out geological investigations three times at the DD phase from 2007 to 2013 after KANSAI joined the Project, in order to advance the basic design of the dam including the quarry and other structures, to estimate quantities, construction cost and schedule.

NNP1 implemented geological investigation in order to identify the weak layers from late 2014 to 2016 during excavation works in the main dam in the Co phase.

2.2. Evaluation of dam foundation rock

2.2.1. In-situ block shear test

Physical properties of the foundation rock were determined based on in-situ block shear test implemented at the test adit. Physical properties of rock mass were evaluated based on Hoek-Brown failure criterion for each part of the foundation rock.

Firstly, In-situ block shear tests are conducted to estimate the shear strengths, which are basically presented by cohesion (c) and internal friction angle (φ). The test results are carefully evaluated taking into account observation of sheared rocks' surface before and after testing, stress-deformation relationship, and the existing test data and references in similar geology. Physical properties of foundation rocks were conservatively evaluated by using the lowest data.

Cohesions are estimated by using the lowest failure point and internal friction angle of 42.5° for CM class and 47.5° for CH class, since plotted data on the diagram are not aligned in a linear shape as shown in Fig. 1.

CM class; τ = 2.01 (MPa) +σ tan 42.5°

CH class; τ = 2.75 (MPa) +σ tan 47.5°

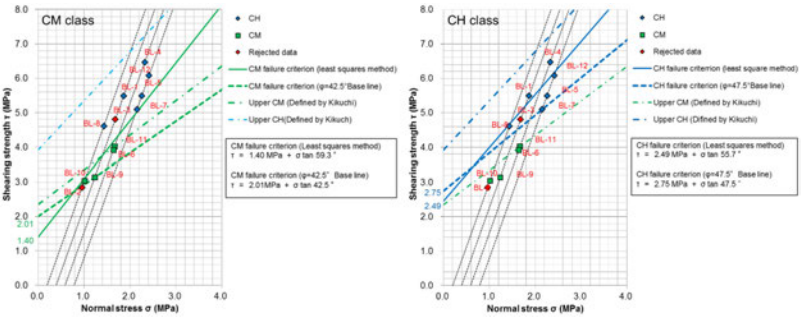

Fig. 1
Shear test results and estimated shear strength
Résultats du test de cisaillement et résistance au cisaillement estimée

2.2.2. Hoek-Brown failure criterion

Spread around European countries, many rock classification schemes have been developed together with civil engineering histories. Of these, two rock classifications systems, the Rock Mass Rating (RMR) system [3], [4] and the Q system [5] have been cited as the leading systems of the world and have been used on many schemes over decades. Both methods tend to empirically incorporate geological features and engineering design in arriving at a quantitative value of their rock mass quality although the differences lie in attempts and concepts. From 2000s onward, the Geological Strength Index (GSI) system [6] has taken the place of the two methods above. The most noticeable feature of GSI is that the rock classification is fused into the original criterion of failure called "generalized Hoek-Brown failure criterion" [7] and realizes a more rationally evaluated shear strength of rock masses without expensive in-situ rock testing.

The rock mass properties and parameters of the Hoek-Brown failure criterion are evaluated based on the above theory. Table 2 presents several cases of linear strength envelopes calculated based on the parameters obtained from field observations.

Table 2

Parameters of Dam Foundation Rock Masses using the GSI System in the Study

ROCK CLASS	CASE STUDIES	ROCK MASS PROPERTIES				PARAMETERS OF HOEK-BROWN FAILURE CRITERION				
		GSI	σ_{CI} (MPA)	MI	D	MB	S	A	σ'_{3MAX}	σ'_{3N}
CH	Average (Ave.)	74	128	17	0	6.72	0.056	0.5	7.2	0.056
	Ave.+1σ	78	175	17	0	7.75	0.087	0.5	7.2	0.041
	Ave.-1σ	70	82	17	0	5.82	0.036	0.5	7.2	0.088
CM	Average (Ave.)	65	65	17	0	4.87	0.020	0.5	7.2	0.111
	Ave.+1σ	72	106	17	0	6.25	0.045	0.5	7.2	0.068
	Ave.-1σ	57	23	17	0	3.66	0.008	0.5	7.2	0.313

In Fig. 2 and the envelopes obtained from in-situ rock tests as follows.

CM class (Ave.); τ = 2.87 (MPa) +σ tan 46.3°

CH class (Ave.); τ = 5.13 (MPa) +σ tan 53.4°

The shaded areas indicate the range of strength envelopes between ±1σ of each rock class. The enveloped zone between ±1σ of CH class is above the envelope of in-situ rock test, thus the shear strength, c and φ, obtained from in-situ rock test is regarded as conservative enough for dam design. On the other hand, in CM class, the envelope of in-situ rock test is over the lower margin of the estimated envelope zone although it gets under the envelope of the average GSI. The result mentions that the shear strength of CM class obtained from in-situ rock test may not necessarily give assurance of the dam safety.

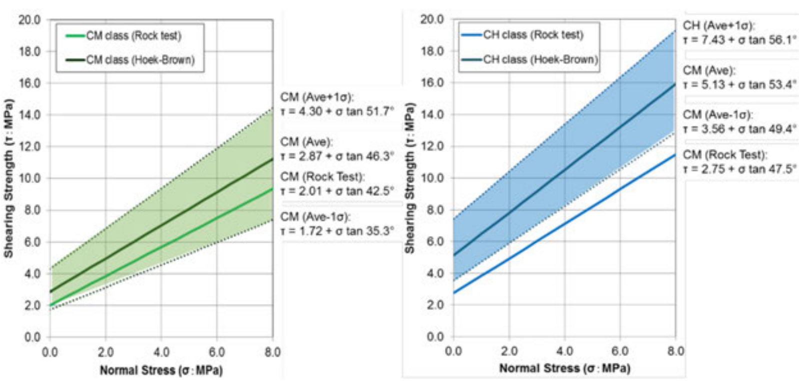

Fig. 2

Comparison of shear strength of CH and CM class

Comparaison de la résistance au cisaillement de la classe CH et CM

2.3. Criteria for Rock classification in the NNP1 main dam site

The rocks were classified by the Central Research Institute of Electric Power Industry (CRIEPI) standard [8]. This standard is usually adopted for dam foundation evaluation in Japan. On the other hand, evaluation by the CRIEPI standard may have a tendency to be ambiguous depending on the geologist because of its subjective evaluation method. This is because the CRIEPI standard requires the description of geological condition by the geologist. Therefore, a universal standard with objectivity for the foundation approval is required.

NNP1 adopted the rock classification by Public Works Research Institute (PWRI) standard [8], by aiming to expel ambiguity derived from the subjective opinion of individuals through geological observation and to more quantitatively classify rock masses into several grades with the combination of basically three important factors which control geotechnical properties of rock mass. The factors which dominate rock mass are very diverse, and noticeable factors are different in accordance with geological conditions and structural design. The PWRI standard has flexibility and is originally defined by compiling findings at all other available projects via geological investigations.

NNP1 extracted three factors of hardness of rock pieces, joint spacing and joint conditions through geological observation on excavation faces of the dam site and defined sub-grades of each factors as shown in Fig. 3 and 4.

Based on the above, NNP1 established the criteria of rock classification in the NNP1 and the existing rock classes in the NNP1 as follows (Fig. 5).

The validity to apply PWRI standard for rock mass evaluation method is checked by putting material properties of foundation rocks obtained through in-situ block shear test on the chart showing the relationship between cohesion and internal friction angle by PWRI.

The surface of the excavated foundation was observed cell by cell, 5 m by 5 m square and the three items of information below were inputted into each cell in order to achieve the process of determination of rock class.

1) Rock class according to the combined matrix of sub-factors for rock mass classification on the excavated rock surfaces
2) Hardness of rock with Schmidt rock hammer
3) Simple elastic wave velocity

Hardness	Description	Representative Rock Face	Unconfined Compression Strength	Rebound of Schmidt Rock Hammer
A	Very Hard, clear or metallic sound by hammer blow. Only possible to crush by strong hammer blow	Fresh micro-conglomerate / Sandstone	> 80 MPa	> 45
B	Hard but slightly dull sound by hammer blow. Possible to crush by hammer blow.	Fresh Mudstone	40 - 80 MPa	35 - 45
C	Medium Hard, dull sound by hammer blow. Possible to crush by hammer blow	Weathered Sandstone	10 - 40 MPa	15 - 35
D	Soft, very dull sound by hammer blow, easy to crush by soft hammer blow, but couldn't shape freely by knife	Weathered Mudstone	1-10 MPa	10 -15
E	Very soft like sandy or clayey soil, possible to dent or penetrate by hammer pick and to shape freely by knife	Cataclastic sandstone	< 1 MPa	Not applied

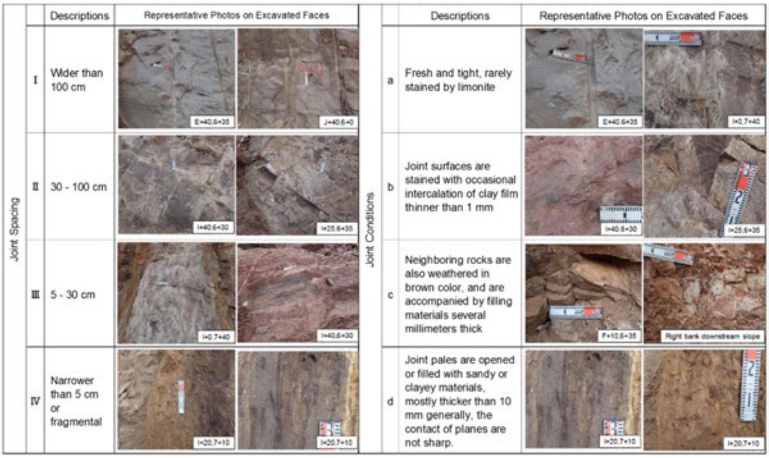

Fig. 3
Sub factor of hardness, joint spacing and joint conditions
Sous-facteur de dureté, espacement des joints et conditions de joint

Hardness		A (Very Hard)				B (Hard)				C (Medium Hard)				D (Soft)				E (Extremely Soft)			
Joint Conditions		a	b	c	d	a	b	c	d	a	b	c	d	a	b	c	d	a	b	c	d
Joint Spacing	I	B	CH			CH	CM			CM	CM										
	II	CH	CM	CM		CH	CM	CL		CM	CL	CL			CL	CL					
	III	CH	CM	CM		CM	CM	CL		CL	CL	CL	D		CL	D	D				
	IV							CL				D	D			D	D				D

Note) Rock classes in colored cells are observable on the excavated faces. Rock classes in white cells are rare combination on the excavated faces

Fig. 4
Combination matrix of sub factors for rock class definition on excavated faces
Matrice combinée de sous-facteurs pour la définition de classe de roche

Rock Class	Description	Combination of Sub Factors ( ): Rare Combination	Rebounds of Schmidt Rock Hammer	Seismic Velocity on Excavated Rocks (km/s)	Shear Strength		Remarks
					c (MPa)	φ (deg.)	
B	Rocks fresh and hard, bedding / joint planes wide in spacing, and tight. No indication of weathering or alteration.	AaⅠa, AaⅠb, AaⅡa, AaⅡb, (AⅠa)	Over 60	Over 5.0	0.288	50	The rock class correspomds to specific stratigraphic facies such as conglomerate and sandstone.These rocks are distributed on a layer structure but not so wide in the main dam site.
CH	Rocks almost fresh and hard, bedding / joint planes wide in spacing, and tight. Slight indication of weathering or alteration.	AaⅢa, AaⅢb, AⅠb, AⅡa, AⅡb, AⅢa	40 - 60	4.0 - 5.0	0.270	47.5	The CH rocks consist of the banks from lower to middle elevation. These rocks are generally distributed on a layer structure.
CM	Moderately hard and moderately jointed or cracked, stained along geologic separation.	(AAⅡc), AⅡc, AⅢb, BⅡb, BⅡc, BⅢa, BⅢb	30 - 50	2.5 - 3.5	0.197	42.5	The CM rocks mainly consist of the main dam site. Occasionally these rosks intercalate week layers of CL class and show closely discontinuous distributions whereas these are widely spreaded on a macroscopic view.
CL	Rock moderately soft but partly hard. Weathering reaches into inner part of rock.	(BⅢc), (BⅣb), (CⅡb), CⅡc, CⅢb, (CⅢc), CⅣb, (DⅡb), DⅢb	10 - 30	1.0 - 2.0	0.068	40	The CL rocks are distributed stratiformly and along specific layers. These rocks generally thicken neighboring to a fractured zone.
D	Very soft, partly decomposed like soil, or blocks are remarkably loosing.	(DⅢC), DⅣc, DⅣd, EⅣd	Less than 10	Less than 1.0	< 0.028	< 20	Fracutured or cataclastic zone mainly comprises D rocks and the rocks are distributed narrowly neighboring to the week zones. Also these are partially distributed around the rim of the dam at higher elevation. The remarkably loosen parts of rock-mass are to be removed.

Fig. 5
Combination Matrix of sub factors for rock class definition on Excavation
Combinaison Matrice des sous-facteurs pour la définition de la classe de rock

2.4. Evaluation of weak layer

The weak layers of total 8 Nos. are located at the dam site which, are FL-A, FL-B, FL-C, FL-D, FR-A, FR-B, FR-C and F-7 as shown in Fig. 6. Weak layers are the geologic separation along the contact bedding planes at the boundary between sandstone and mudstone layers during tectonic movement including folding, and afterwards the layers were suffered from weathering. The dam stability may not be secured due to these weak layers. Therefore the property of the weak layers are evaluated as follows.

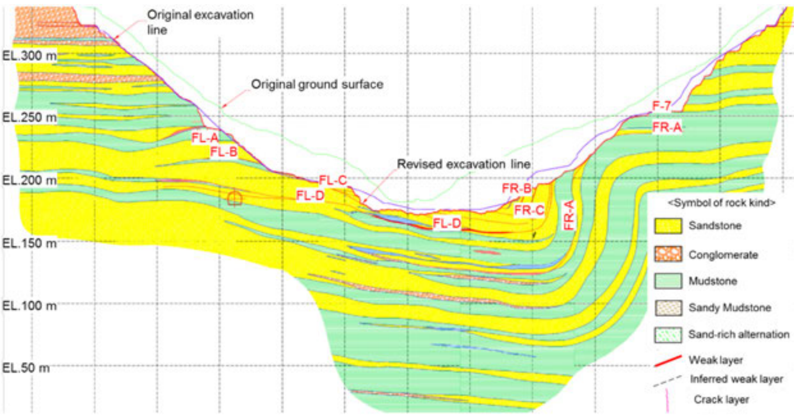

Fig. 6
Geological profile along the dam axis with crack map (Rock kind)
Profil géologique le long de l'axe du barrage avec carte de fissure (type de roche)

2.4.1. *Shear box test*

Physical properties of weak layers were determined based on shear box testing of the disturbed samples. In addition, the physical properties of weak layers were checked by estimation from liquid limit and plastic limit testing.

The strength of weak layers should be evaluated considering the condition after impounding. Weak layers are consolidated completely and excess pore water pressure does not occur. Therefore, shear box tests were conducted with drained condition after consolidation. Weak layers are consolidated slowly in parallel with the dam concrete construction. Therefore, the speed of consolidation of weak layers after impounding is slower than two times 90 % of the consolidation period. Generally, the slower the shear speed becomes, the weaker the strength of material becomes. The shear speed of weak layers after impounding is also very slow and like a creep failure so that the shear speed is controlled at 0.02 mm/min which is restricted by the capacity of the testing machine. The maximum compressive stress after impounding is over 3.0 MPa so that the pressure of shear box testing is set to be greater than 2.0 MPa which is also restricted due to the capacity of the testing machine. Residual strength of weak layers is adopted as the design strength of the weak layers, because the dam should be stable even in the condition that the weak layer is at yield due to earthquake or other load. In this case, cohesion is lost. Therefore, cohesion of weak layers is not considered for design.

The weakest weak layer at either abutment area is FR-A. The result of shear box testing of FR-A is shown in Fig. 7. The tests were conducted by varying the shear speed from 0.02 mm/min to 0.05 mm/min. Test results show that the strength of weak layers with 0.02 mm/min shear speed is lower compared to the strength of weak layers with 0.05 mm/min shear speed. The strength of the weak layer is c = 0 MPa and φ = 20.1°. Other weak layers had stronger shear strength than FR-A. Therefore, design values of weak layers at the abutments area of c = 0 MPa and φ = 19.0° were adopted by considering safety margin.

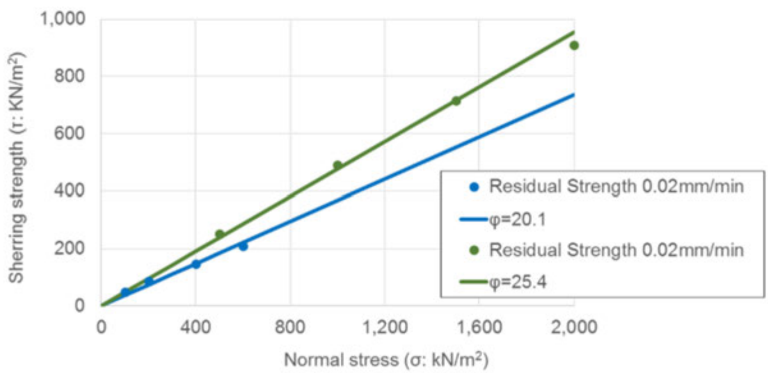

Fig. 7
Residual strength of FR-A
Résistance résiduelle de FR-A

The weakest weak layer at the riverbed area is FL-D. The result of shear box test of FL-D is shown in Fig. 8. The strength of weak layer is c= 0 MPa and φ= 23.6°. The peak strength is higher by 3.7° than residual strength. FL-D has higher strength than FRA. The evaluation of the weak layers at the riverbed area is very important, because large load due to the high water pressure is imposed on the foundation rock.

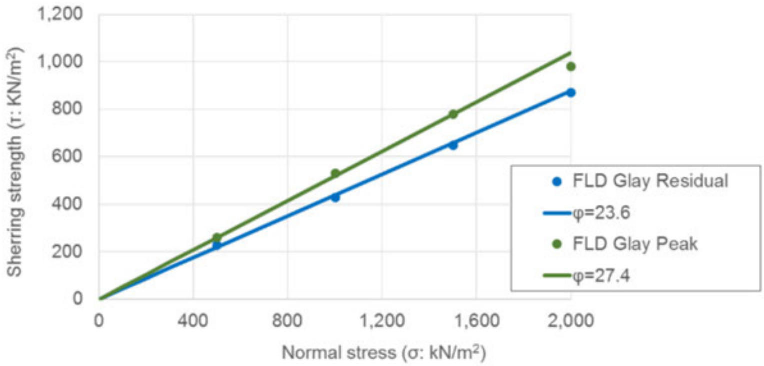

Fig. 8
Peak and Residual strength of FL-D
Peak et résistance résiduelle de FL-D

2.4.2. Estimation of material properties from plasticity index

Generally, the strength of soil and the plasticity index are correlated. When the plasticity index is small, the strength of soil is high. Test samples were extracted from four location along FL-D. The estimated strengths of FL-D from the plasticity index is shown in Fig. 9. The results show that the estimated internal friction angle is higher than 25.0° at every location.

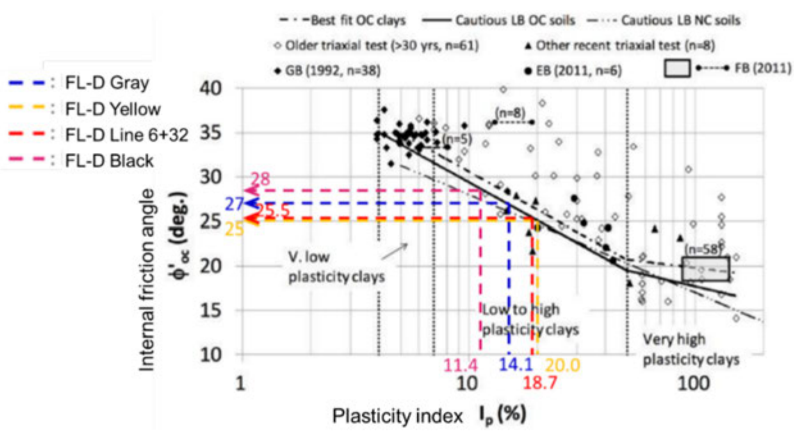

Fig. 9
Results of estimation of material properties from plasticity index
Les résultats de l'estimation des propriétés des matériaux de l'indice de plasticité

2.4.3. Estimation of strength of weak layer considering component ratio

FL-D consists are not homogeneous. The total strength of the FL-D can be estimated from the component of weak layers. FL-D consists of 18 % sandstone without fine materials and 82 % soil materials. Therefore, the total strength of FL-D is estimated to be c= 0 MPa and φ= 25.5° in proportion to each component. The strength of FL-D considering component ratio of weak layers is shown in Table 3.

Table 3
Strength of FL-D considering component ratio of weak layers

GEOLOGICAL CONSISTS	SUBJECT OF AREA	COMPONENT	INTERNAL FRICTION	TEST
Sandstone	16 m	18 %	33.0°	In-situ bock shear test
Mudstone	0 m	0 %	-	-
Soil	74 m	82 %	23.6°	Shear box test
Total	90 m	100 %	25.5°	Component

2.4.4. Clay material test

XRD analysis was conducted in order to verify that these fine particle fractions in weak layers do not include any swelling or clay material such as, for example Smectite which might significantly degrade physical properties of soil materials. Results of XRD analysis of drilling core D-49 shown in Fig. 10 show that crystalline particle structure is low due to a high degree of weathering since the peak value is not large. The obvious peak value of Quartz and Mica were observed, while the small peak value of Vermiculite or Smectite. The former is not swelling clay minerals, but the latter is swelling clay minerals.

Next, detailed analysis for clay materials by adding ethylene glycol (EG) treatment was conducted. As a result, the clay materials were specified to be Vermiculite. Therefore it was quantitatively verified that highly swelling clay minerals such as Smectite did not exist in the clay materials.

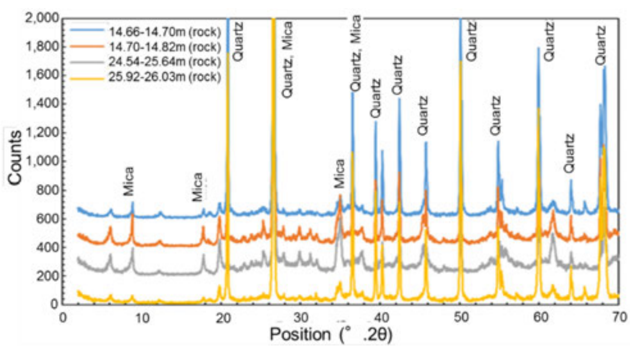

Fig. 10
XRD patterns for D-49 core
Modèles XRD pour le noyau D-49

2.4.5. Continuity of fracture

Many drilling cores were examined. It was observed that fracture can be divided into three types.

Type I: Homogeneous fracture of the mud stone within the sand stone layer
Type II: Highly weathered fracture of mud stone with sand stone or conglomerate
Type III: Leaching Calcite fracture of red color mud stone with void

Continuity weak layers such as FL-D are classified Type II. Drilling surveys were conducted at the riverbed area as shown Fig. 11. A fracture was found in D-54 drilling core which is located at the boundary of mudstone and sandstone layers below FL-D. On the other hand, the part of D-50 drilling core which is located around the boundary of mudstone and sandstone layers below FL-D is intact. These fractures are classified as Type III and fractures do not seem to be connected to nearby cores. If these fractures continue at the riverbed area, the dam safety is not secured without countermeasures being undertaken. To investigate the continuity of fractures, Computed Tomography (CT) scanning was conducted for drilling cores along the boundary of mudstone and sandstones layer below FL-D as shown in Fig. 12.

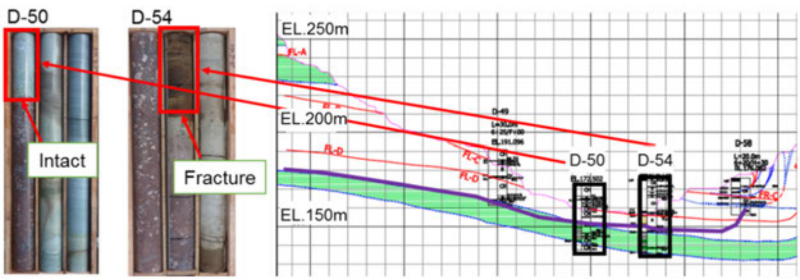

Fig. 11
Drilling core and location of drilling (Dam axis)
Noyau de forage et emplacement du forage

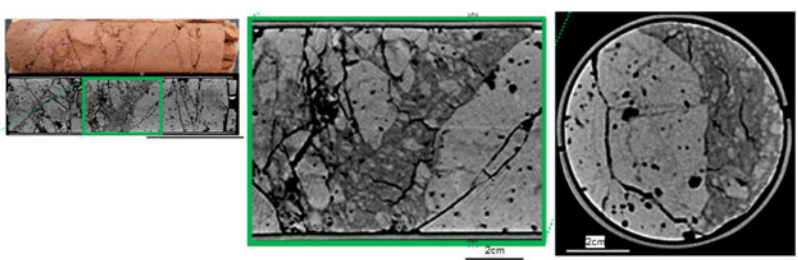

Fig. 12
CT images of core
Images CT du noyau

The dark color indicates the low density part. There are many 5 mm diameters voids. 20 mm wide fracture with fine particles and many pebble fragments were observed. However, rearrangement of fragments with fine particles were not observed. It means that this fracture was not made by a dynamic movement such as slip or sliding. In addition, this fracture is classified as type III. From the above reasons, these fractures seem not to be continuous.

2.5. Geological Evaluation

The geoligical plans showing rock class and kind at the dam site, ratinge of rock class and kind, and the geological profile with crack mapping along the dam axis are shown in Fig. 6, 13 to 14. The ground surface of the main dam site is covered by talus deposit underlain by stratified structures of sandstone and mudstone which are categorized as CM to CH in rock mass classification. Orientation of all rock mass discontinuities on the both abutment and riverbed including the fold zone and shear key structures have been studied in stereo net analysis as shown in Fig. 15. As general the bedding planes on the both abutment are moderately dipping with 8° in downstream direction and 8° in riverbed direction. And around the middle of the dam axis the dip angle of the bedding plane is 15° to 25° in the same direction. No outstanding fault has been observed and a box fold has been formed at the steep slope area.

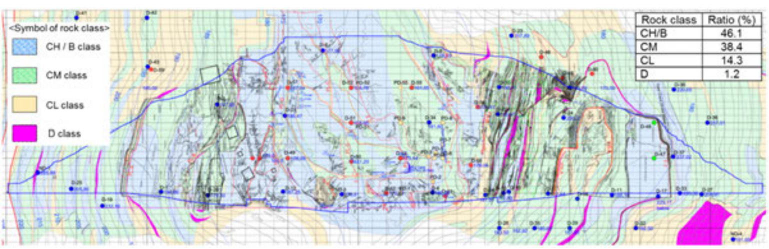

Fig. 13
Geological plan (Rock class)
Plan géologique (classe de rock)

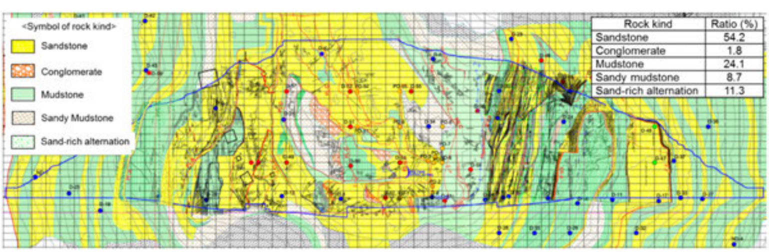

Fig. 14
Geological plan (Rock kind)
Plan géologique (Rock kind)

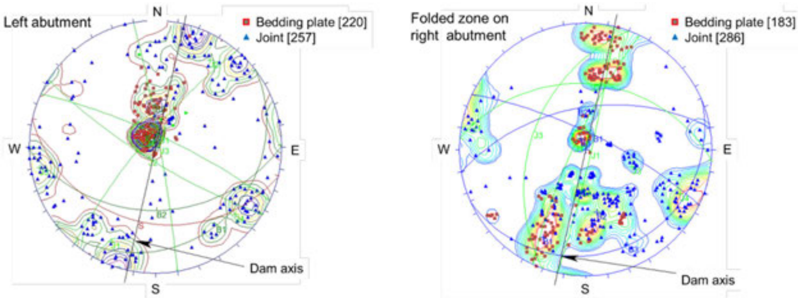

Fig. 15
Distribution of dip angle of crack at the left bank abutment and folded zone
Répartition de l'angle d'inclinaison à la rive gauche et de la zone pliée

2.6. Characteristics of geology

1) FS phase

Special attention was given to selecting a dam site and a dam type, by considering rock class and rock mass condition. Through the geological studies, some characteristic geological the defects in stratified structure of sandstone as a massive rock and mudstone as weak rock were observed. Continuous cracks generally emerge in a direction perpendicular to bedding planes due to solidification shrinkage of massive sandstone and actually some vertical deep openings were observed on the foundation rocks at the possible dam sites. Highly weathered-fractured zones were also detected around the folded zone over a width of around 100 m on the right abutment.

2) DD phase

The intensive geological investigation was conducted along the dam axis at the selected dam site. The drilling investigations revealed deep and highly permeable zones in the fractured zone created by the fold formation on the right abutment and some permeable zone along the bedding plane on the left abutment and the riverbed. The mechanical properties of the outstanding fractured zone designated as F-7 was studied in the exploratory adits dug on the right abutment. The shear strengths of foundation rocks and modulus of deformation of rock mass were also examined in the same adits and the initial design of rock mass classification was conducted to apply to dam stability analysis.

3) Co phase

According to geological investigations in the previous two phases, further studies on the fractured zones and open joints were conducted, which was considered as additional investigation to provide accurate and sufficient geological and geotechnical input to the dam stability analysis. The method of drilling using foam with some modification such as using triple core tube and diamond bit system in combination with a highly effective foam generation device was applied in order to extract 100% core. High quality core samples with 100% core recovery made it possible to observe full cores and to classify fractured zones in detail (Fig. 16).

Those core samples were subjected to CT scanning analysis to distinguish fractured zones to be recognized as continuous weak layers that could affect the dam stability from only highly jointed zone. Some minor fractured zones exist below FL-D which has a prominent deformation structure and continuous profile. The deformation structure of each zone is obscure and fine materials due to the weathering process are not observed in joint openings. FL-D was identified as the deepest continuous weak layer to be considered in the dam stability analysis.

The shear strength of highly weathered and decomposed material in the fractured zone was examined through a shear box tests by using the core samples. The shear strength of c= 0 MPa with φ= 19.0° represent the strength of fine materials in all continuous weak layers to be integrated to the analytical model.

Fig. 16
High quality core samples
Échantillons de base de haute qualité

3. GEOLOGICAL RISK

The geological risk was re-evaluated based on the newly obtained geological information and by focusing on weak layers which might much affect the dam stability, incremental increase in project cost and delay of the construction period.

3.1. Geological risk of Hydropower BOT

The project had been developed with Engineering, Procurement and Construction (EPC) Full Turnkey basis during the FS phase to DD phase but did not reach agreement. After that, NNP1 changed the contract type from EPC Full Turnkey basis to Bill of Quantity (BoQ) basis before staring DD phase.

Geological risk associated with the development of a hydropower project is considered as the most common issue in almost every kind of infrastructure project. In hydropower projects, the geological risk is to be considered to be particularly relevant and the greatest amongst all the various risks and more so than for any other infrastructure project. Substantively, hydropower projects are subject to consideration of how to address the geological risk well. In this chapter, the detailed analysis of the geological investigation and evaluation discussed in the previous chapter is introduced as a way of dealing with the geological risk in the hydropower project.

3.2. GEOLOGICAL RISK

The dam was re-designed to have large margin. The dam height was enlarged from 148 m to 167 m, after it was found necessary to further excavate foundation rock and to install a shear key for ensuring dam stability. As a result, the volume of excavation and RCC are increased by 21 % and 15%, respectively. In addition, the cost of geological investigation, excavation and RCC were increased by 16 % more than that in the DD phase. The comparison of the dam dimensions between in the FS phase, DD phase and Co phase is summarized in Table 4.

Table 4
Comparison of dam dimensions with geological information

ITEM / PHASE	FEASIBILITY STUDY	DETAILED DESIGN	CONSTRUCTION
Dam foundation rock	CM, CH class	CM, CH class	Almost same as DD phase of project
Characteristic Geological	Folded zone Weathered fractured zone	Folded zone One (1) weak layer	Folded zone **Eight (8) weak layers** (horizontal and vertical)
Dam type	RCC gravity dam	RCC gravity dam	**RCC gravity dam with shear key structure**
Dam height	151.0 m	148.0 m	**167.0 m**
Dam crest length	600.0m	530.0 m	535.5 m
Increment rate of geological investigation cost	-	-	**64 % increase in geological investigation than DD phase (5 % increase in total cost[*1] than DD phase)**
Excavation volume and increment rate	1.52 Mm^3	1,650,218 m^3	2,003,896 m^3 **21 % increase in DD phase (37 % increase in total cost[*1] than DD phase)**
RCC volume and increment rate	2.6 Mm^3	2,035,397 m^3	2,340,979 m^3 **15 % increase in DD phase (58 % increase in total cost than DD phase)**
Assumed geological risk (contingency)	(Total cost: increase about 10 %)	Increase in the excavation, RCC, grout and measures for weathered and fractured zones (Total cost: increase about 10 %)	**Increase in quantity of excavation, RCC, grout and measures for weak layers (Total cost: about 16 % increase)**
Others	EPC Full Turnkey	BOQ	BOQ

Note: Total cost = Geological investigation + Excavation + RCC

3.3. Risk reduction measures and results

Geological investigations and dam designs conducted in the FS phase, DD phase and the Co phase are analyzed in detail as follows:

- Minimized amount of the geological investigation during the DD phase gave an appropriate design of general excavation line without significant change except additional excavation of the weak layers exposed in the riverbed (Fig. 6).
- Prompt and systematic geological and geotechnical studies were conducted for horizontal weak layers, FL-A, FL-B, F-7 and FR-A, encountered on the both abutments during dam foundation excavation works. In addition, the Contractor's voluntary acceleration made it possible to minimize delays in construction terms. As one of the most significant issues in dam foundation excavation works, installation of the shear key structure in the riverbed was found indispensable for ensuring dam stability. The exact dimension of the shear key structure depends on shear strength of weak layers, their location and continuity. The Contractor's engineers joined the dam engineers and geologists of the Owner side on site and discussed details of the excavation profile every day and consequently the most suitable dimensions of the shear key structure could be designed and constructed thereby minimizing surplus construction time.
- As discussed above, sophisticated drilling methods were applied during the Co phase in order to have higher quality investigation to examine the complicated geological condition containing very weak materials and to obtain more accurate distribution of geological defects in a timely manner. A Japanese drilling company was selected to extract full cores for thorough observation of cores and all core specimens and sampled materials were transferred to Japan for laboratory tests.
- The total cost for geological investigation, dam foundation excavation and RCC placement volume increased by 16 %. The incremental cost was arranged through contingency allowed for construction cost, escalation clauses, contingency for escalation clause and further economical design of dam and its appurtenant structures. Construction cost contingency is usually a key factor in the Co phase. Its amount depends on a number of risk factors of the project and especially on the geological structures. It is important to estimate the amount of contingency referring to past experience of projects having similar geological condition.
- The core sample investigation conducted in the DD phase was based on the core samples taken by a local company. From a total 42 Nos boreholes 3,240 m total length, investigation detected only one fractured zone as a future weak layer. Cost minimizing within the limits that general geological features can be obtained is necessary for the project development. However it is indispensable for geological risk control to have enough knowledge and consideration of possible geological defects to be specified in the geological structures with detailed planning of further investigations to be conducted in the next phase.

- Since it is difficult to employ a site investigation drilling company in the countries of South East Asia with the capability of very high core recovery, assignment of a skilled drilling company, from Japan for example, might result in cost savings and moreover geological risk reduction. It is perhaps superfluous to say that this needs to be done within the limits of budget in the DD phase.
- The target and cost of geological investigation by assuming geological risk in each phase are as follows:

  1 FS phase.
     - To evaluate feasibility of the project by selecting the dam site and dam type and to predict the geological risk.
     - Comprehensive geological site investigation drilling and non-destructive testing conducted by the owner.
     - This cost is limited and to be incorporated in the development cost.

  2 DD phase.
     - To finalize dam design and estimate the cost and schedule and to study possible countermeasures against geological risk.
     - Supplemental geological investigation such as additional drilling and in-site laboratory tests are implemented by the project company with high experienced engineers and geologists.
     - This cost is to be incorporated in the project cost and managed by the project company.

  3 Co phase.
     - Supplemental geological investigations such as using advanced drilling techniques and sometimes more sophisticated, scientific testing methods, can be implemented by the project company with special team, if geological defects are found.
     - This cost is to be covered by contingency.

4. CONCLUSION

Based on above discussion on the geological evaluation and risks encountered in the hydropower project development, we conclude that:

- The geology at the dam is formed by alternating mudstone and sandstone layers, which have a gentle inclination towards downstream. The weak layers are distributed along the boundary of mudstone and sand stone layers, and the pen-sized cracks have developed across the weak layers. On the right bank, there is a remarkable folded zone and parts of the layers at the folded zone show nearly vertical inclination.
- The material properties of foundation rocks are properly estimated based on in-situ block shear testing and Hoek-Brown failure criterion.

- The material properties of the weak layers are properly estimated based on shear box testing, the component ratio of soil materials and plasticity considering the condition after impounding.
- It was quantitatively verified that swelling clay minerals such as Smectite did not exist based on XRD analysis.
- The fractures along the boundary of mudstone and sandstone below FL-D do not continue based on the drilling investigation and CT scanning survey.
- The Project had been developed with EPC Full Turnkey basis during the FS phase to DD phase but did not reach agreement with possible bidders since their bid prices to address geological risk were too high to make the Project feasible, practical or realistic. Consequently the Owner changed its direction to carry the geological risk by itself.
- The geological risk can be controlled and reduced when the geological investigation is properly and sufficiently conducted with well-organized quality and balanced quantity in each phase of FS, DD and Co.
- It is indispensable in a large-scale hydropower project with BOT scheme to implement a detailed study, by referring to past projects which have similar geological features, if possible, and to have capable geologists and dam engineers who have sufficient knowledge and experience with geological survey teams and equipment to detect the possible geological risks to be encountered during dam foundation excavation works and to solve these issues with less additional cost and within a certain period of term.
- The power utility company which has initiated more than a few hydropower projects from the initial stages and which owns and operates them over a long time as one of the executors of hydropower development has the experience of controlling geological risks of projects during implementation, though geological risks vary project by project.

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