Appendix 1

Consideration on Geology of Bagong Damsite

1. Geology of the dam site.

1.1 Geological stratigraphy

- The Middle to Late Neogene Wonosari Formation and the Paleogene Oligocene to Early Neogene Mandalika Formation are distributed around the dam site (Figure-1).
- The Wonosari Formation consists of limestone*, sandstone and mudstone, while the Mandalika Formation consists of breccia, lava, tuff, sandstone and siltstone. The relationship between these formations is unconformity.
- The Wonosari Formation observed at the dam site consists mainly of alternating layers of coralline limestone and calcarenite, partly intercalated with thin layers of mudstone (Table-1).
- After the start of construction, it was found that “Colluvium” was thickly distributed on the slopes on both banks of the dam site (Figure-2). However, the properties of colluvium differ between the right and left banks. The colluvium classified on the right bank is very loose rock mass, the origin of which is estimated to be creep or landslide (Photo-1, Photo-2).
- Overlying the colluvium classified on the left bank is a relatively thick layer of volcanic ash classified as “clay”. This volcanic ash is estimated to be the product of the Kawi-Butak volcano, located approximately 20 km north of the dam site.

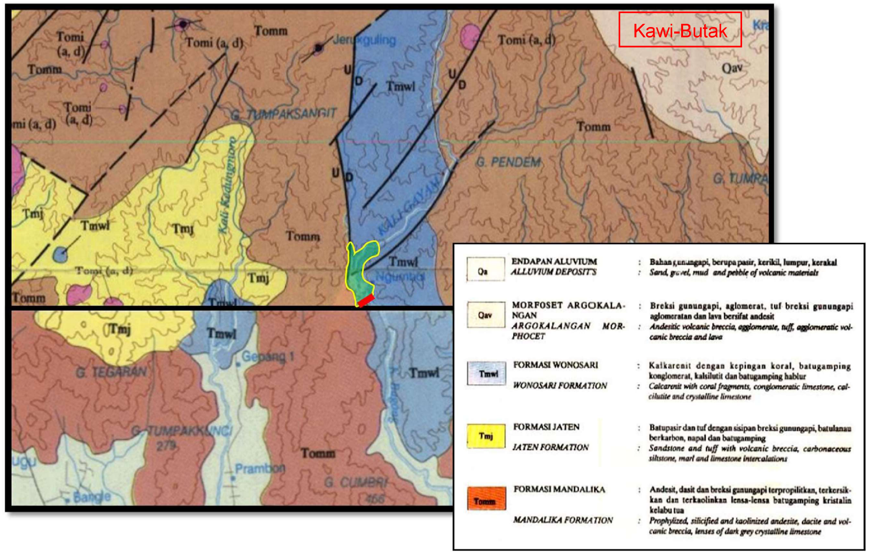
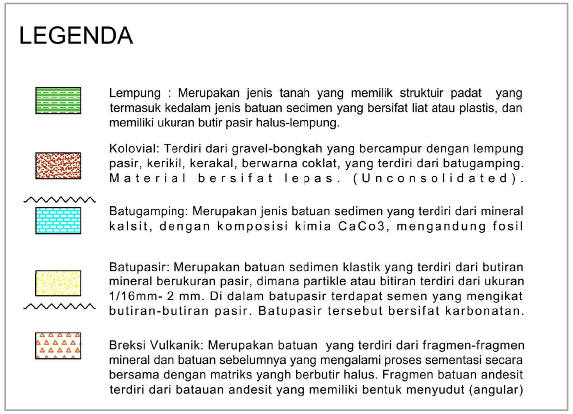


Figure-1 Regional geological map.

Source: Material provided for site inspection

*Limestones are divided into two main categories, crystalline and amorphous, and amorphous limestones are divided into three categories based on the size of the clastic particles. Those containing gravel-size particles are classified as calcirudite, those composed of sand-size particles as calcarenite, and those dominated by silt-size or smaller particles as calcilutite.

Table - 1 Geological composition of the dam site



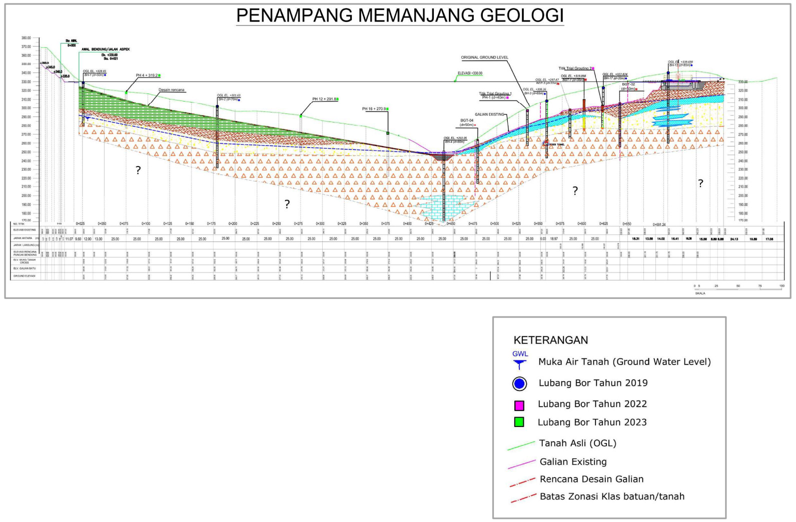


Figure-2 Geological cross section of the dam axis.

Source: Material provided for site inspection

a) Limestone

b) Mudstone

Photo-1 Colluvium on the Right Bank

The grey layer rocks are dominated by mudstone and the brown layer rocks by limestone. The colluvium is weathered but retains its stratigraphic structure.

Photo-2 Colluvium on the Left Bank

The colluvium is a gravel-mixed sediment, with a high proportion of limestone as gravel.

1.2 Geological Structure

- According to the regional geological map, several northeast-southwest and north-south faults have developed around the dam site (see Figure 1).
- High-angle fractures have developed in the limestone and sandstone in the Wonosari Formation distributed at the dam site. Fractures are developed in the thin claystone layers, which have partial slickensides (Photo-3).
- Based on the above, it is estimated that the Wonosari Formation around the dam site has been subjected to folding. The geological conditions may also have been a factor in the formation of thick layers of colluvium.
- The dip of the strata observed during the field visit is almost horizontal to less than 10°. However, according to the geological cross section of the dam axis shown in Figure-2, the distribution altitude of limestone is slightly different between the right and left banks, which may indicate the distribution of old faults in the riverbed (which can be confirmed by excavating of the riverbed area).
- As limestone is the dominant gravel in the colluvium on the left bank, it is estimated that limestone layers are widely distributed upslope.
-

Photo-3 Claystone Layer in the Wonosari Formation.
Fine fissures are developed and partially fissure surfaces have slickensides.

2. Engineering Geology

2.1 Causes and Characteristics of Colluvium

(1) Right Bank

1) Causes of Colluvium

- The retention of geological structures within the colluvium (see photo 1) suggests that the colluvium has crept or moved as a mass. The slope is close to the summit, indicating that the distance over which the mass has moved is short (Figure-3).
- Based on the material provided for the site inspection and interviews with the site engineer, the distribution area of the colluvium is estimated to be approximately the area indicated by the red circle in the Figure-3.
- According to Google Earth satellite imagery, no landforms suggestive of land sliding (e.g. slip cliffs, gentle gradients at the head) are recognized around the slope, and the short distance of movement is estimated to be rock mass creep.

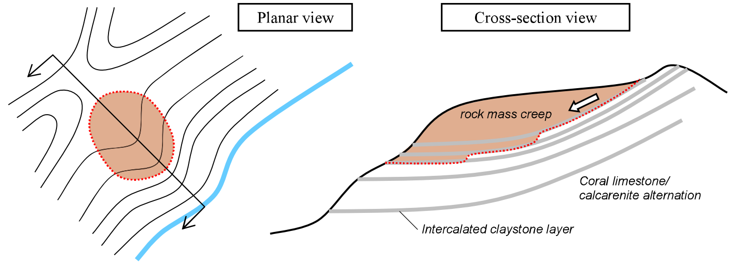


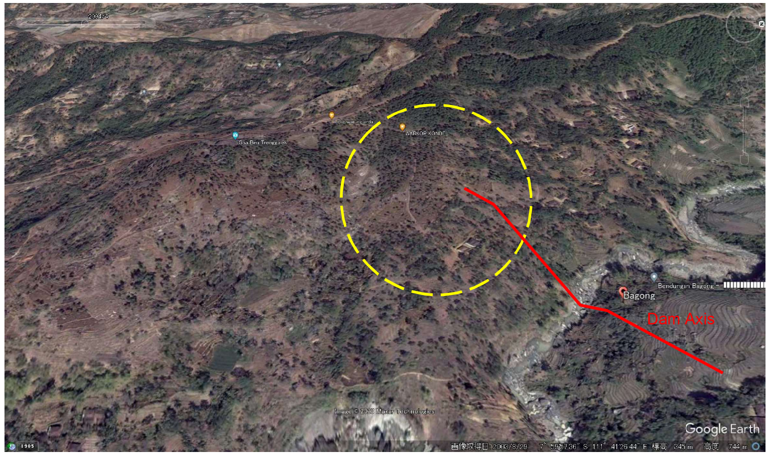
Figure-3 Schematic diagram of rock creep on the right bank

2) Characteristics of Colluvium

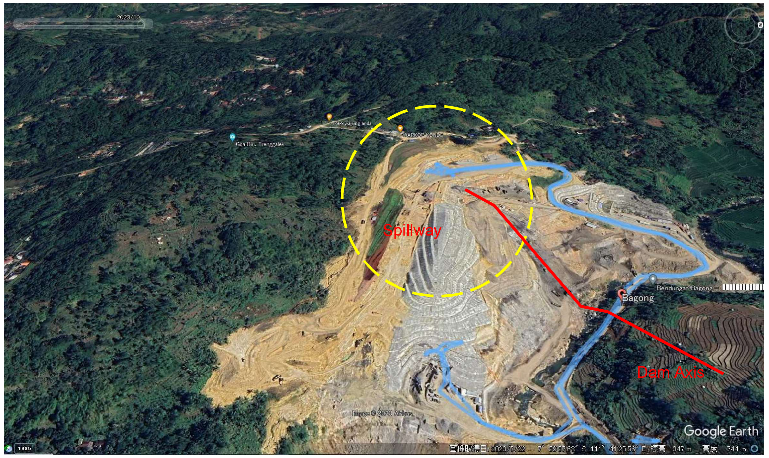
- The main difference between a colluvium and a landslide is whether it has a slip surface (slip layer). Landslide surfaces generally consist of a clay layer and are impermeable. As springs are observed on slopes close to the underside of the colluvium, it is very likely that a slip surface exists. However, as the distance travelled by a landslide is not long enough for a slip surface to develop, the slip surface is likely to be formed in an existing claystone layer.
- In general, the slip layer consists of highly plastic clay. Due to the shear surfaces (foliation) caused by creep and sliding, they are anisotropic in strength and have a lower strength than normal clays. During the site inspection, highly plastic clay was observed at a depth of 17-19 m in borehole BGT-5 (Photo-5).
- In the site plan, the strength of the colluvium is set at $c = 24.5$ kPa and $\varphi = 16°$ for both the left and right banks. These values generally correspond to the strength of softer clay layers. However, clays in slip layers may have an internal friction angle of less than 10° due to the development of shear planes.
- In general, rock masses that have undergone creep and sliding have open fractures and high permeability. Lugeon test results at the site indicate that 67 Lu (in the order of 10^{-4} cm/s) have been obtained in collapsed soils. However, local permeability may be much higher. There may be cavities inside the mass.

Photo-5 Core samples from borehole BGT-5 at depths of 15-20 m

Highly plastic clays are recognized at depths of 17-19m.



a) Satellite image taken on 29 August 2003



b) Satellite image taken on 22 July 2023.

Note: The blue line is the GPS trajectory at the time of the site inspection on 12 October.

Photo-4 Satellite image of the left bank of the dam site.


Photo-6: Slope left between the spillway and the embankment.

The slope of the cut is 1:2.0 and soil nails have been used.

In the long term, there is also concern about the destabilization.

(2) Left bank

1) Causes of Colluvium

- According to Google Earth satellite imagery, steep slopes or cliffs can be seen above the left bank slope (Photo-7). It is estimated that the sediment generated by the slope failure near the summit has repeatedly moved and accumulated due to colluvium and debris flows, forming a thick layer of colluvium below the slope, and the area where colluvium is distributed forms a gentle slope.
- The above colluvium is covered with volcanic ash. According to the geological cross section of the dam axis in Figure-2, the distribution of the layer is thicker upslope and reaches more than 20 meters.

2) Characteristics of Colluvium

- Since the colluvium formed as described above does not have a sliding layer. Therefore, an arc-shaped slip surface can be adopted in the slope stability analysis.
- Based on the properties of the colluvium observed during the site inspection (abutment downstream of the embankment), it is estimated to have a bearing capacity as the foundation.
- The colluvium has a gravel to clayey soil ratio of approximately 1:1, and the average permeability is estimated to be on the order of 10^{-4} cm/sec. However, piping holes are generally present in colluvium.
- On the engineering geologic map, the collapsed soil and volcanic ash layers are classified as D class. However, the strength and permeability of the D class is not clear in the materials used for our site inspection (the parameter is necessary to conduct the stability analysis of the dam).

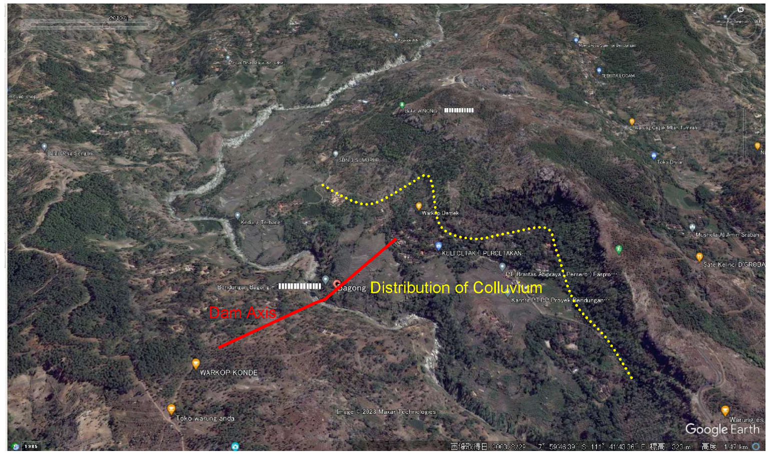


Photo-7 Satellite image of the right bank of the dam site, 29 August 2003.

2.2 Tunnel construction

- A ceiling collapse occurred during the construction of the diversion tunnel (Photo-8). The strata observed at the outlet of the diversion works tunnel were calcareous sandstone (calcarenite), partially intercalated with thin layers of claystone (Photo-9).
- The geological structure is almost horizontal and the claystone layer above is brittle with well-developed shear planes (see Photo-3, Photo-10). The collapse points inspected on site show the distribution of a thin layer of claystone layers, which is estimated to be the cause of the ceiling collapse. Due to the relationship between the height of the tunnel and the geological structure, the claystone layer is distributed just near the ceiling.

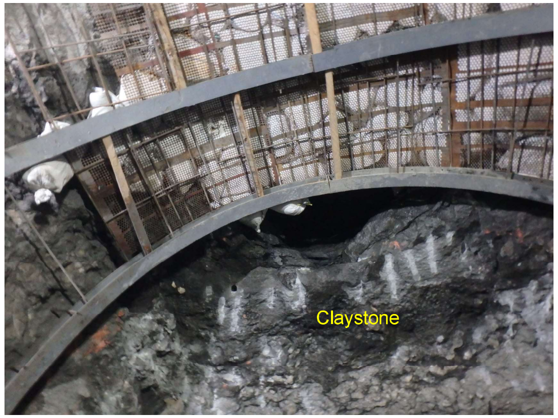


Photo-8: Collapsed portion at the ceiling of the diversion tunnel.

Photo-9 Rock mass conditions near the outlet of the diversion tunnel (1).

Photo-10 Rock conditions near the outlet of the diversion tunnel (2)

Attachment 2

Suggestion on Treatment of Colluvium at Bagong Damsite Right Abutment and Spillway Foundation

JICA Dam Construction Advisors
Hiroshi Shimizu
Naoya Mizuno

1. Issue

i. The foundation of the right dam abutment and spillway consists of colluvium (creep zone?), so there is a risk of deformation due to overload by embankment, earthquakes, or water pressure.

ii. When the foundation is deformed, a space will be created between the spillway concrete and the foundation because the bottom of spillway concrete is fixed with concrete piles.

iii. This space can become a water path and cause an increase of water leakage and a piping.

Dam is a structure subject to keep high water pressure, so this situation is quite danger.

My highest concern is inequal deformation of foundation at the joint between the spillway concrete and the dam body (core foundation). When the dam embankment is commenced, dam foundation will settle (together with the plasticity Secant Pile? I think secant pile is elastic, not plastic). On the other hand, the spillway concrete is supported by the concrete piles so there is no settlement following with the settlement of foundation.

An inequality deformation will occur at this portion. This space can become a water path and cause an increase of water leakage and a piping.

2. Suggestion

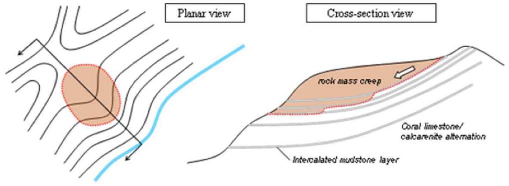
Followings are our suggestion.

(1) Geological Analysis

To decide the final design, you need more geological survey and detailed analysis on geological structure and properties of the colluvium. Currently, we evaluate that the layer identified as colluvium is creep zone. In case the layer is creep zone, it is necessary to design the countermeasures that takes a movement of this layer into consideration.

In our geological analysis based on the given information and data by the Project, and interpretation of topographic map/satellite images, the layer calling as colluvium is analyzed as a rock creep zone (refer to Figure A1 in Annex 1), from the following reasons:

- Several thin clay layer is confirmed in the boring photos of GBT-05 and GBT-02 (see Photo 1 & 2 in annex 1 (A1)), and the lowest layer is probably distributes as shown in Figure A1,
- The upper layer of the thine clay layer is obviously loosened and showing characters of colluvium (eg. distribution of open cracks, weathering condition and remaining of geological structure in the sliding mass), and,
- No topography suggestive of land sliding around the right bank slope (e.g. slip cliffs, gentle gradients at the head and the short distance of movement, etc.) (by reading of Google Earth satellite image) .



Schematic diagram of rock creep on the right bank

Proposed additional geological study/survey is attached in Anex 1.

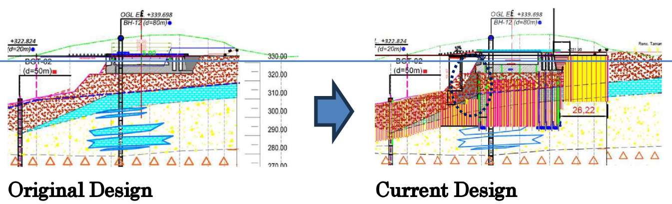
(2) Comment on Current Design (Separate Wall between Dam Body and Spillway)

Currently, separate wall is proposed between dam body and spillway by the Project. This is one of the ways to mitigate the risk.

(a) Design

Based on the drawings, the structural features are;

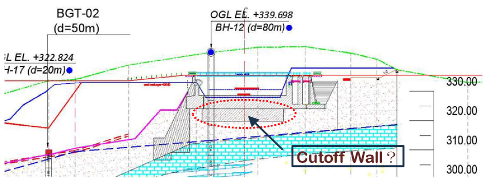
- ✓ Foundation : Limestone Layer
- ✓ Height of Wall : 27.5 m
- ✓ Front Slope : H:V = 1:0.5 (Spread Foundation)



Note:

a) It makes sense to extend the Secant Pile to the sandstone layer as a water stop measure considering the permeability of limestone.

b) There is something drawn, which is connecting to secant piles, along water stop line in the drawing, but no details. Is this a concrete cutoff wall? if it is cut off wall, the both side shall be connected to impermeable structure/foundation.



(b) Comments on the Separate Wall between Dam Body and Spillway

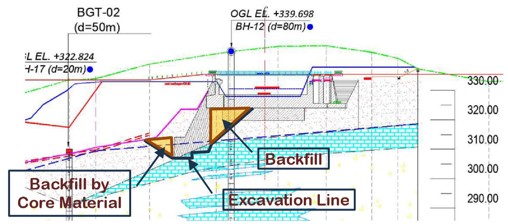
(i) Foundation of the Separate Wall

It is supposed that the wall was designed on a limestone as its foundation under the condition that the limestone would have sufficient bearing capacity since. However, the concrete piles for spillway support have penetrated through the limestone layer and into the sandstone layer. It is not clear why the design is.

Since the height of the wall is 27.5 m, it is recommended to confirm the strength of limestone and modify the design of the wall foundation or pile foundation, if necessary.

(ii) Care of Backfill between Cut Slope and Wall

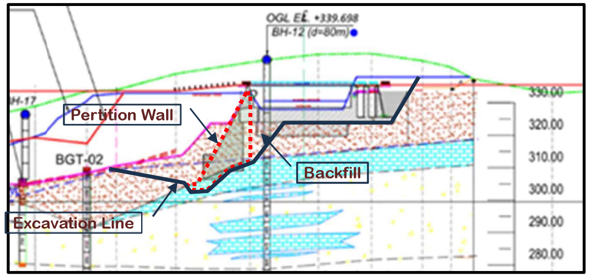
When constructing the wall between the core and spillway, it is necessary to excavate deeply in section, and the excavated area shall be filled with backfill material. That means creating a weak point in the foundation of the spillway. Backfill material will settle due to consolidation, but spillway concrete does not settle because it is supported by concrete piles. Therefore, there is only one point to ensure water stoppage: the connecting point of the Secant Pile and the spillway concrete. It is not sure how reliable a watertightness at connection between the secant pile and concrete spillway will be, and whether it will be able to ensure watertightness for long term. Once piping occurs, the backfill materials will flow out and the wall tilts, leading to the dam braking.



(iii) Shape of Core Foundation

If deep excavation and remain the Colluvium, it is necessary to pay attention a design of connection point of the wall and core foundation. Generally, core foundations are excavated smoothly without leaving noticeable protrusions, but in the current design, the toe of the concrete protrudes, so it is recommended to lower the foundation another around 1 meter and embed it in Limestone.

(iv) Proposal of Design Modification



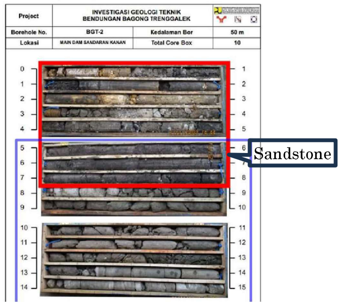
(3) Alternative Plan

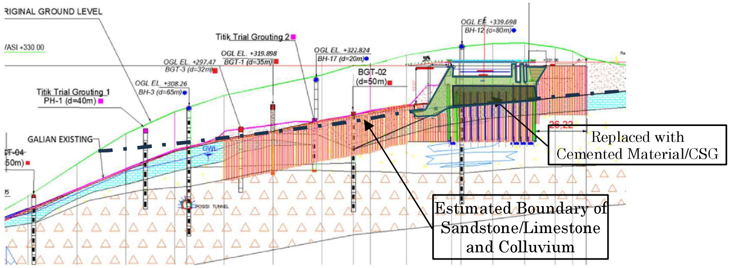
(a) Alternative I (Remove Colluvium)

As for another practical use of the original design, which can be secure a more safely condition, remove all the colluvium on the foundation of core and below spillway, and replacing concrete such as soil cement or CSG.

The deepest additional excavation depth is more than 20 m based on the geological map. However, judging from drilling core phot (BGT-2) which is located the deepest position, sandstone layer good for core foundation can be confirmed at the 8 m from the excavation surface. Therefore, only additional 8 m deep excavation will be required even remove all colluvium.

BGT-2 (50 m)





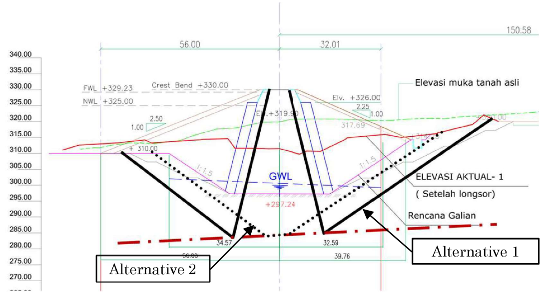
Spread Foundation of Spillway and Separate Wall

As for the excavation plan at the core foundation, there is two alternatives.

✓ Alternative 1 is foundation excavation in accordance with core design.

✓ Alternative 2 is just secure minimum core width at the bottom. In this case, it is anticipated that an arch action occurs and brake core by hydraulic fracturing. Therefore, the core material must select (mixed

material of cores material, fine sand and clay) to minimize the consolidation after embankment.



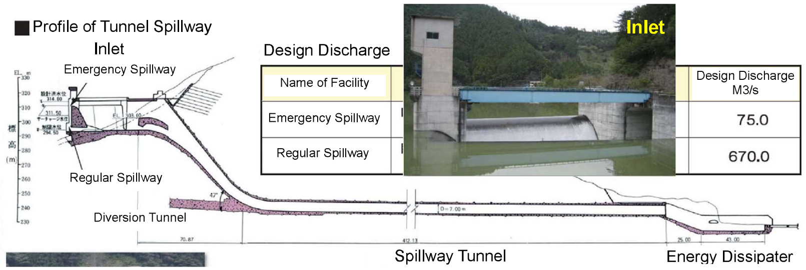
Excavation of Core Foundation

It is recommended that the Engineer reconfirm the geological conditions around the right bank dam abutment and spillway, and then make a comparison study on the construction cost, safety, and workability of the excavation method and the Secant Pile method.

(b) Alternative II (Change spillway type)

When the construction of current design of spillway cannot be adapted, one of the solutions is to apply tunnel spillway with a morning glory type or bathtub type overflow section.

Figure below shows an example of bathtub type tunnel spillway.



Name of Facility	Design Discharge M3/s
Emergency Spillway	75.0
Regular Spillway	670.0

Tunnel Spillway (Bathtub Type: Arima Dam in Japan)

Appendix 1:

Proposed Additional Geological Study and Survey

Based on the results of geological analysis, following additional geological study and survey are proposed.

Location of proposed additional borings is as shown in Figure A2.

1. Updating of Geological Map at Damsite

Since the additional borings has been conducted during construction stages, but there are several points which have not been referred. The review of geological analysis should be conducted, and the geological maps be update using latest data/information.

2. Dam Foundation

Based on core photos of GBT-05 and GBT-02, the thickness of limestone layer is thinner than that of showing in the geological map and boundary between sandstone and volcanic breccia may be located upward.

In order to set a more reasonable design of the Secant Pile, it is recommended to recheck the all BGT borings and past drilling logs, and then conducting additional boring surveys (B-1, B-2), if necessary.

3. Separate Wall Foundation

The separate wall with 27.5 m high will stand independently during construction stage. After backfilling behind the wall, unidirectional earth pressure will load to the wall (no counter earth pressure from core), and large principal stress will occur at the toe of wall.

It is recommended, therefore, to confirm the ultimate bearing capacity of the foundation rock (limestone) as well as to recheck the design and construction procedure of the wall considering construction stage. Also, it is recommended that to conduct a drilling survey (B-2) and physical tests to confirm the depth (elevation) and strength of the foundation rock, if necessary.

4. Right Rim Grouting Area

Required area of curtain grouting (or secant piles) on the right rim and below spillway cannot be identified because there is no geological data (core boring). It is estimated that limestone which is high permeability In the right abutment of the dam, so it is anticipated that the seepage from the abutment.

It is recommended to check past drilling logs and to conduct additional boring surveys (B-3 to B-5), if there is no geological information to confirm the geological structure and permeability on the abutments.

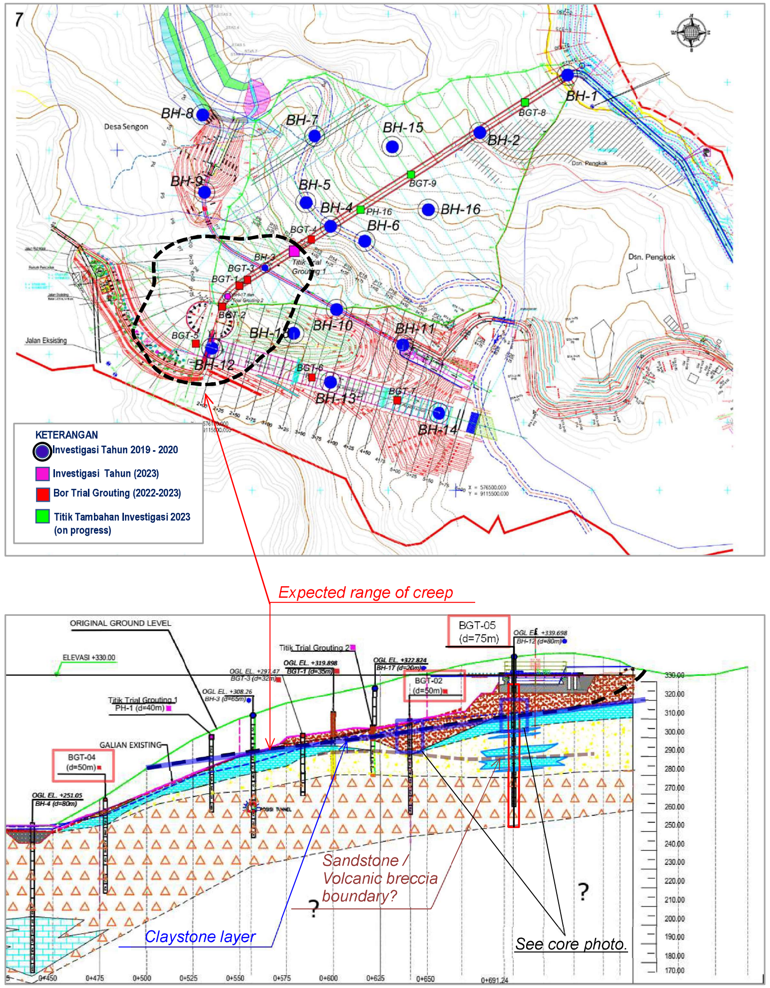
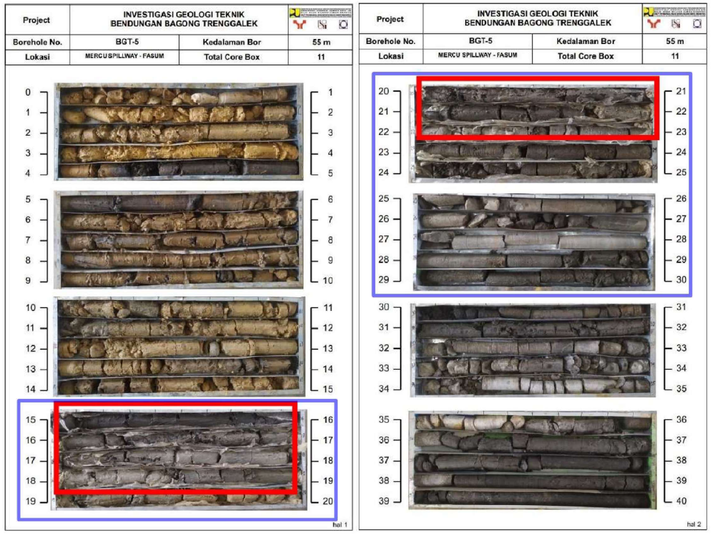


Figure-A1 Geological Map of the Right Abutment & Spillway of the Bagong Dam

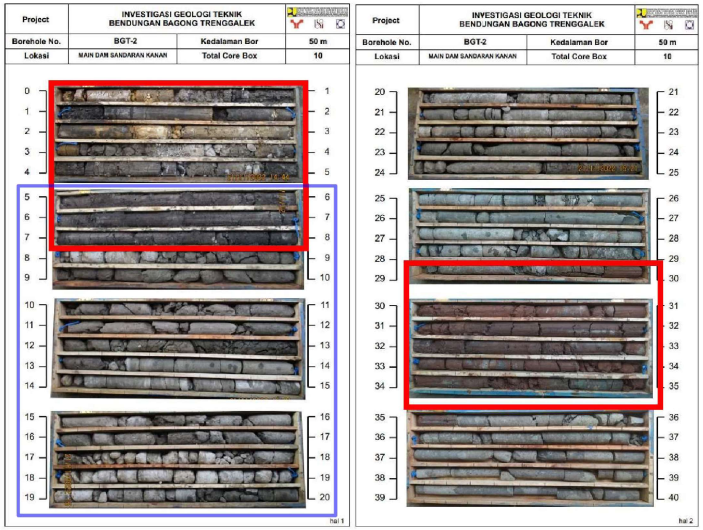
Phot 1

- **BGT-5 (55/75 m)**



Phot 2

- **BGT-2 (50 m)**



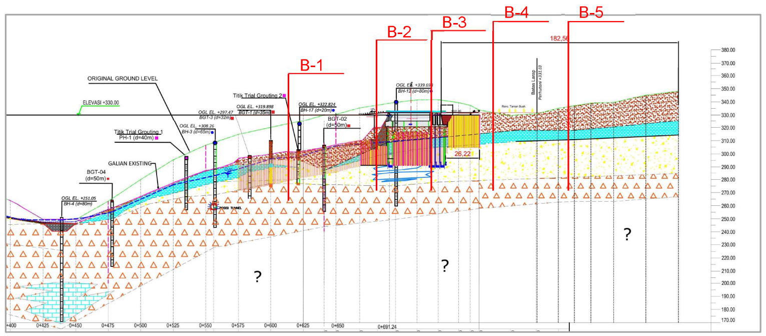


Figure-A2 Proposed Locations for Additional Drilling

Site Inspection Report

Hiroshi SHIMIZU
JICA Dam Construction Advisor

Name of Dam : Bulango Ulu Dam
Date : February 5-6, 2024

1. General

1.1 Bulango Ulu Dam

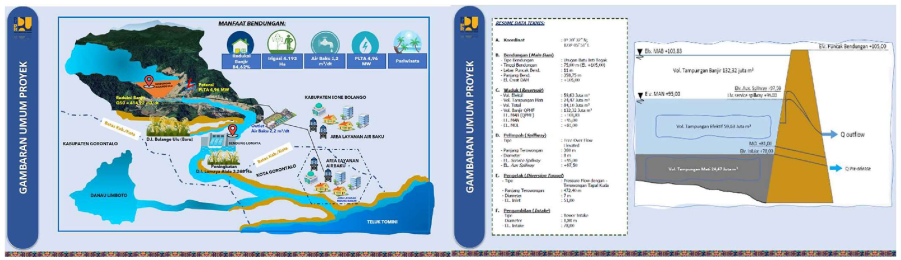
The studies for Bulango Ulu Dam have been carried out by CIDA (Canadian International Development Agency) in 1991 - 1999 and by JICA in 2001 - 2002. Both studies had recommended the construction of a reservoir in the area.

The Bulango Ulu dam has for the following purposes:

1. Irrigation Development
2. Hydroelectric Power Plant (PLTA)
3. Flood Control (Flood control)
4. Provision of raw water for clean water in Bone Bolango Regency and surrounding districts, and
5. Others (fisheries, tourism, sports facilities and environmental improvement)

In particular, the spillway has a unique structure that is rare in Indonesia. A tunnel spillway is applied, and a part of the overflow section of the emergency spillway is lowered to control the design flood as a regular spillway for flood control.

(1) Location and Purpose of Bagong Dam



(2) Regional Geology

Bedrock : Granodiorite Igneous Rock Unit

Rock Color : Fresh, whitish gray, weathered brownish white

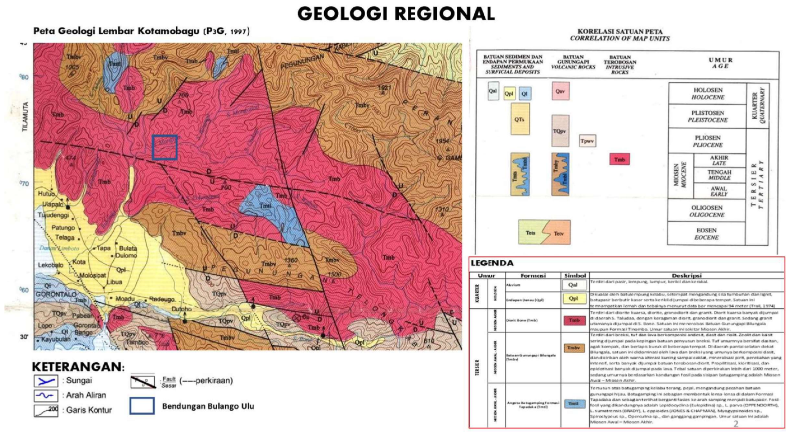
Granularity : Faneric

Mineral Composition: Orthoclase, Biotite, Hornblende and Quartz

Rock Type : Acid Igneous

Granodiorite is a plutonic igneous rock, which is formed from an intrusion of silica-rich magma that cools in batholith below the earth's surface. Granodiorite can be exposed on surfaces after uplift and erosion.

Based on regional geology of Kota Mobagu sheet (T. Apandi and S. Bachri. 1997), Bulango Ulu dan located at Bone Diorite Formation (Tmb) that consist of quartz diorite, diorite, granodiorite and granite. Quartz diorite was found extensively in Taludoa River area with diorite, granodiorite, and granite variations. Granite was mainly found in Bone River area. This unit intruded the Bilungala Volcanic and the Tinambo Formation. The age of this unit is Late Miocene approximately.



The Foundation of Bulango Ulu Dam intrusive igneous rock (Granite, Granodiorite, diorite and andesite). The rock foundations for Bulango Ulu dam are classified into 3 classes, they are:

- ✓ CL Class : Lugeon=11-44 and qu 10-80 kg/cm2
- ✓ CM Class : Lugeon=10-21 and qu 80-400 kg/cm2
- ✓ CH Class : Lugeon=4-10and qu 400-700 kg/cm2

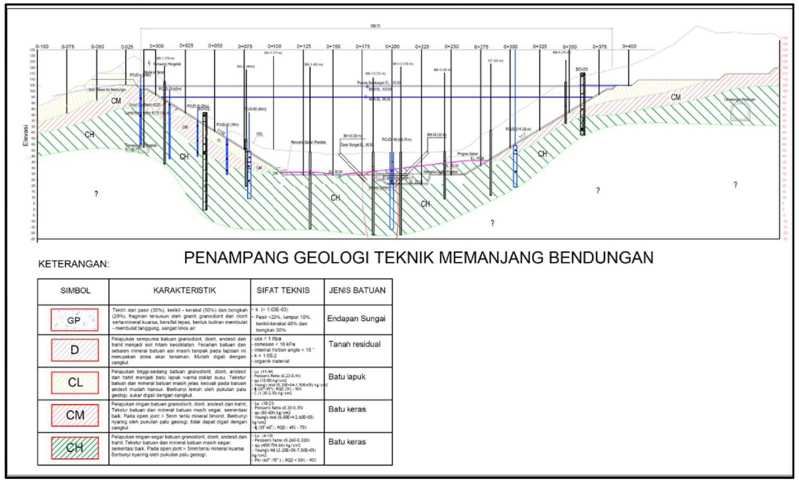
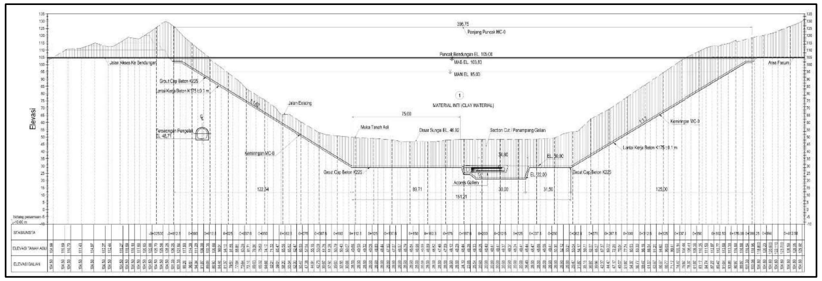
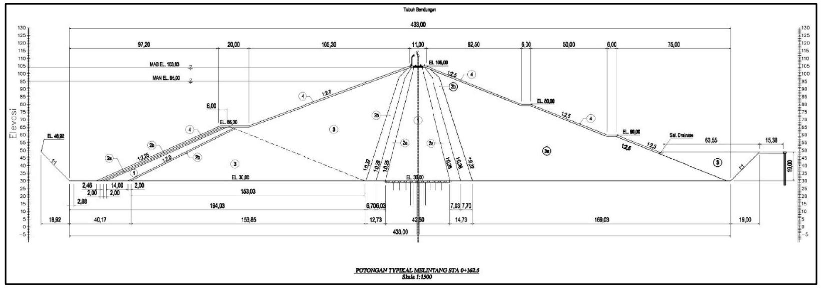
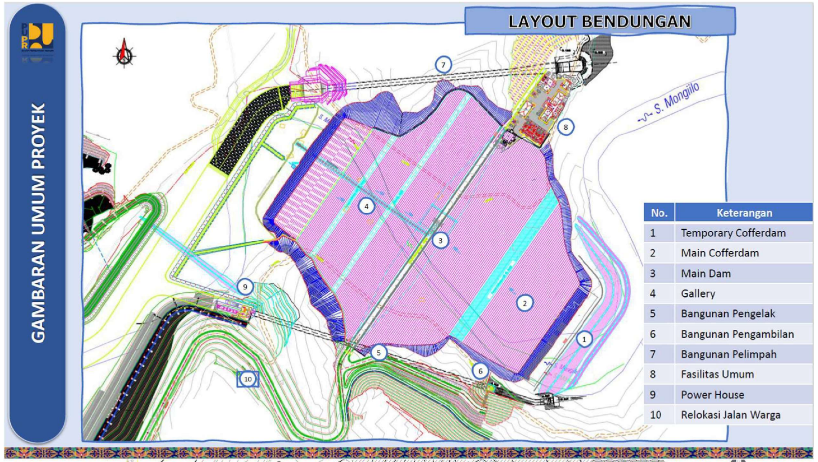


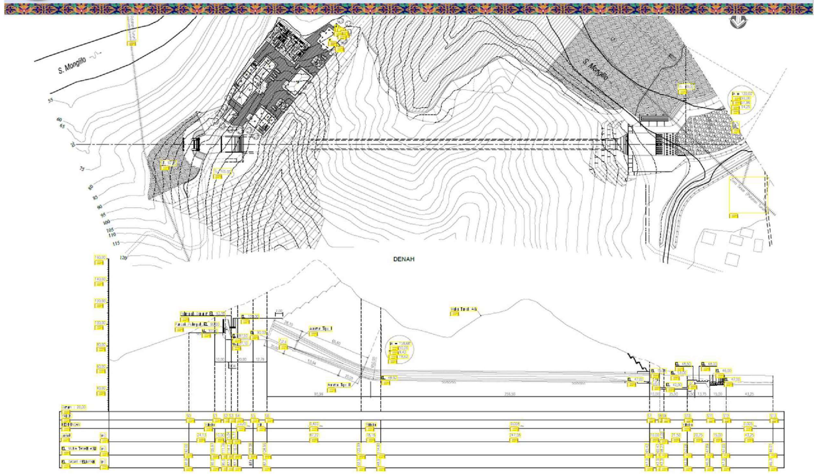
Figure 1 Engineering Geological Profile of Bulango Ulu Dam

(3) Drawings of Plan-Profile-Section and Intake Structure









2. Progress/Schedule as of February 2024

At the time of the inspection, foundation excavation has been almost complete. However, even though the embankment works for dam body has not yet commenced, tryal impounding is scheduled at six months later, in September 2024.

[to be confirmed]

3. Issues Discussed

3.1 Discussed Points

(1) Fault on Dam Foundation

At the dam foundation, “Fault” and “Ancient River Trail” were observed. The rock condition around the fault are highly weathered and fractured, with some clay material.

At present, although the details of them are not confirmed because the foundation excavation has not been completed, it is supposed that these issues will not be a critical probrems considering the scale of foult.

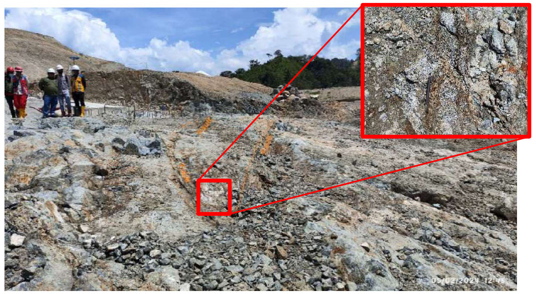


Figure 2 Fault Crossing Core Foundation

(2) Spillway Inlet and Outlet

The spillway of Bulango ulu dam is tunnel spillway with Faults and Intrusion were observed around the spillway excavation.

Surrounding intrusion rock (dike), the base rock (Granodiorite) has been weathered and decompressed then It's like sand. It is necessary to protect the excavated surface not to develop the weathering.

Figure 3 Inlet (left) and Outlet (right) of Spillway of Bulango Ulu Dam

(3) Core Material

The Core material of Bulango Ulu dam will be supplied at several borrow pits. The material is weathered granite/granodiorite, which are widely distributed at the site. However, the clay content is limited to a depth of several meters, and deeper than that, it is slightly weathered sand. Issues remain on the development area and amount to be secured.

(4) Rock Material

The rock material of Brango Ulu Dam is taken from the quarry area. The material is granite/granodiorite whose main minerals are plagioclase and orthoclase. The compressive strength of granite is over 700kg/cm2.

Weathering seems to progress more easily around intrusive rocks such as andesite, so it is necessary to segregate such rocks.

3.2 RECOMMENDATIONS

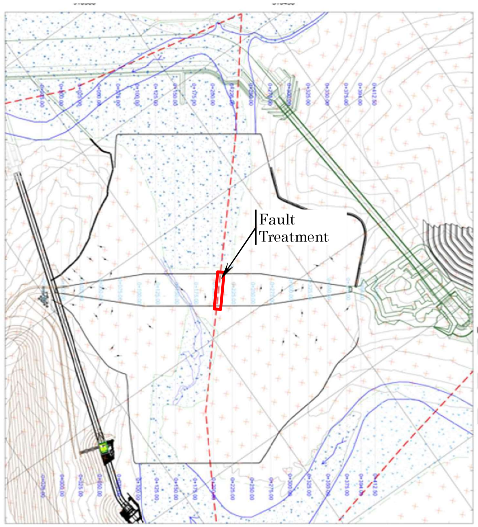
(1) Fault on Dam Foundation

In general, faults in fill-type dam foundations will not pose a major problem as long as watertightness is ensured, as the fill dam body itself has the ability to follow the deformation of the foundation. However, it will need to be replaced with concrete if significant displacement is expected at the fault zone after embankment, in particular there are concrete structures on the core foundation, such as galleries, spillways, grout caps, etc., . The concrete replacement for fault treatment will be the shape shown in the figure below, similar to the case of gravity dams.

Since the core foundation of Bulango Ulu Dam will be covered with grout cap concrete, it is recommended that to conduct a trench excavation and replace it with concrete along the fault at the core foundation area, if necessary.

In addition to the faults, at the location of the ancient river, it is recommendable to excavate until hard rock is found and then dental concrete is applied so that the surface is flat.

For foundation treatment, "Design Standards No. 13 Embankment Dams Chapter 3: Foundation Surface Treatment Phase 4 (Final)" U.S. Department of the Interior, Bureau of Reclamation July 2012 can be referred.



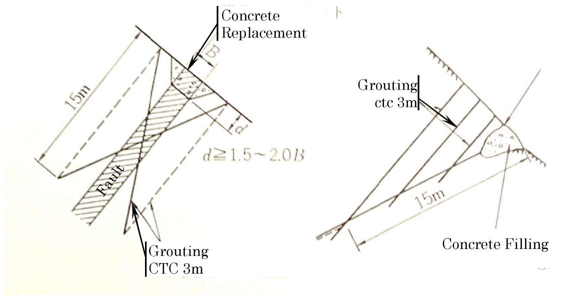
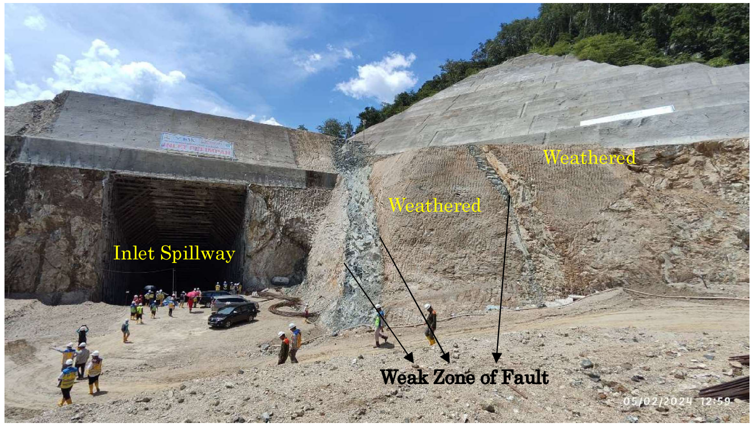


Figure 4 Fault Treatment of Dam Foundation (Sample of Dam in Japan)

(2) Spillway Inlet and Outlet

Surrounding intrusion rock (dike), the base rock (Granodiorite) has been weathered and decompressed then It's like sand. It is necessary to protect the excavated surface not to develop the weathering.

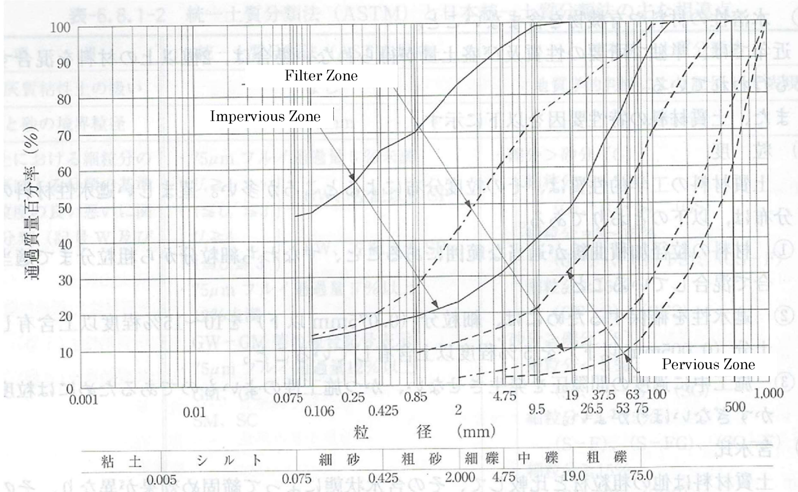
At the inlet of the tunnel, a fault in the granitic rock is clearly visible. Although the fault has been backfilled by the intrusion of more alkaline rocks, the boundary between the two rocks appears to have a fracture that becomes a weak zone. Improvements to the location are required by grouting to minimize the permeability value of the rock to prevent side seepage through the weak zone.

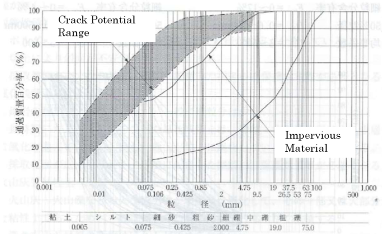


Weathered and fault at the inlet of spillway tunnel

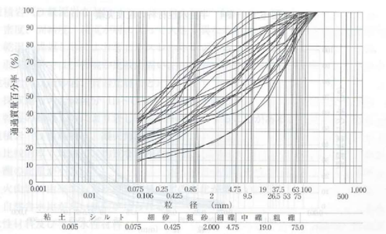
(3) Core Materials

- ✓ When the zoning of core and applied materials have been changed, dam stability shall be recalculated, and its calculation sheets shall be submitted to BTB applying the tested parameters during construction.
- ✓ Since The fine material (under #200 (0.074mm)) contents used for the core embankment reaches more than 75%, this condition will be difficult to compact, therefore the material intake must be deepened so that the coarse material is also mixed so as to increase the angle value (phi). The consultant shall trial embankment with proper composition taking into account of (at least 50-60% fine material composition). The workability of compaction should be confirmed through the tryal embankment at site.
- ✓ Following figures showing Japanese guideline prepared by Ministry of Agriculture, Forestry and Fishery (MAFF) and Ministry of Land, Infrastructure, Transport and Tourism (MLIT). The guidelines show that fine material contents (under #200) are set between 15% to 50%, considering watertightness, potential cracks and workability for compaction.





Potential Range of Crack Occurrence



Grain Size Distribution of Core Material (Actual Data of Dams by MAFF Japan)

Guideline of Grain Size of Rock-fill Dam Embankment Materials (MAFF Japan)

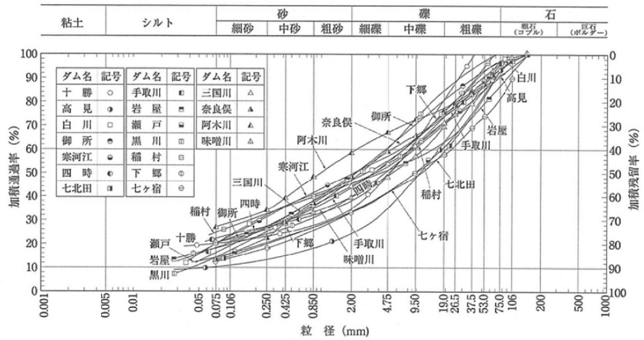


図 25-3 土質材料の粒度分布例

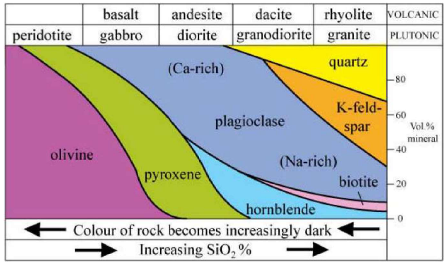
Sample of Garin Distribution of Core Materials in Japan

Note : The core materials used in MLIT Japan are coarse-grained which are securing a required watertightness and good workability for compaction. Material are obtained from mainly weathered rock, weathered residual soil, and terrace deposits.

- ✓ The specification of embankment materials have not yet been established while the trial embankments to determine it also not been carried out. Trial embankment for all materials shall be carried out immediately using equipment to be used in the field.
- ✓ To speed up the works, a larger equipment such as 20-ton compaction roller may be used.

(4) Rock Materials

Granodiorite/granite is composed of coarse grains and this rock, although hard in general, will weathered quickly to form sand, in particular near the intrusive rocks. During the mining of rock materials, such rocks should be segregated properly for the applied rockfill material.



Mineral assemblages in igneous rock

This plagioclase mineral is not good enough to withstand weathering so that it will usually disintegrate faster. This condition is not good as riprap material, especially in the upstream part of the dam that is in direct contact with water. Therefore at the upstream location of the dam, the use of this rock is to be avoided as riprap because weathering will be faster besides that replacement is also relatively difficult due to the inundation of the dam. While in the downstream part although weathered quickly but the replacement is relatively easier because it is not impounded by water.

(5) Others

- ✓ Modifications of design and construction method should be recorded in the form of a matrix of changes and submitted to the BTB and DSC, for Risk Management in future.
- ✓ The work schedule should be rechecked and substantiated considering the actual field condition to pursue the impounding schedule that is planned to be implemented in August 2024. In the rescheduling, the state of resources, equipment and financial conditions shall also be considered.
- ✓ Since outlet facility is installed using diversion tunnel which consists of only one tunnel, it is necessary to pay attention to the installation schedule and safety of the hydromechanical equipment connection.
- ✓ An early warning device for rain is needed to be installed around the embankment site during the embankment implementation and also a fast communication system in case the Ru value in the core zone almost reaches the safe limit to stop the embankment addition temporarily.
- ✓ The Project office requested Sample of Payline. The actual payment method (specifications) and over-brake pay-line (Bili-Bili Dam and Jatibarang Dam) were attached in Attachment 2.

Design of Core Width of Rockfill Dam

1. General

Core embankment works for rockfill dams poses challenges such as difficulty in securing a sufficient amount of core material and constraints on construction due to rainfall and temperature, and it has become a critical-path for the entire dam construction. Therefore, it is required to conduct a rational reduction in core width in design.

The current core width is empirically determined by setting the top width of the core to 4 to 6 m due to construction constraints, and setting the upstream and downstream slope of the core to 1:0.2. Some dams have adopted a core slope of 1:0.16 to 0.17, which is slightly steeper than 1:0.20, in combination with filters that meet Sherard's limit filter criteria[1] 2), which stipulates stricter piping conditions than current filter standards, but this is not based on core stability analysis. On the other hand, based on the analysis of past cases of seepage failure, it has been determined that the ratio of core width (B) to dam height (H) must be at least B/H = 0.25 or more.

In view of this current situation, it is necessary to clarify the following points.

① The minimum core width (upstream and downstream slope of the core) in the design.

② Change of shape of core (change of core slope in core zone) in case it becomes clear that the amount of core available is smaller than design during the construction stage.

③ Importance of consideration of hydraulic fracturing of the core.

In the "EM 1110-2-2300 Earth and Rockfill Dams – General Design and Construction Considerations" (Department of the Army, U.S. Army Corps of Engineers, 31 July 1994), the core width of rockfill dams is explained as below:

(2) The core width for a central impervious core-type embankment should be established using seepage and piping considerations, types of material available for the core and shells, the filter design, and seismic considerations.

In general, the width of the core at the base or cutoff should be equal to or greater than 25 percent of the difference between the maximum reservoir and minimum tailwater elevations. The greater the width of the contact area between the impervious fill and rock, the less likely that a leak will develop along this contact surface. Where a thin embankment core is selected, it is good engineering to increase the width of the core at the rock juncture, to produce a wider core contact area. Where the contact between the impervious core and rock is relatively narrow, the downstream filter zone becomes more important. A core top width of 10 ft is considered to be the minimum for construction equipment. The maximum core width will usually be controlled by stability and availability of impervious materials.

2. Rational Design Method for Core Width

The Civil Engineering Research Institute of the Ministry of Land, Infrastructure, Transport and Tourism Japan had conducted a "Study on Rational Design Method for the Core Width of Rockfill Dams", and the design method is being presented. The flow of the rational design method for core width based on the core's hydraulic fracturing resistance is shown below.

1. Physical and mechanical tests will be conducted on the core and filter materials, and physical property to be used for embankment and impounding analysis will be set. Regarding the core material, a hydraulic fracturing test shall be conducted to evaluate the constants "m" and "n" that define the hydraulic fracturing resistance pf = m･ σ 3+ n based on the empirical formula obtained from the results of many past hydraulic fracturing tests. If necessary, perform a tensile strength test

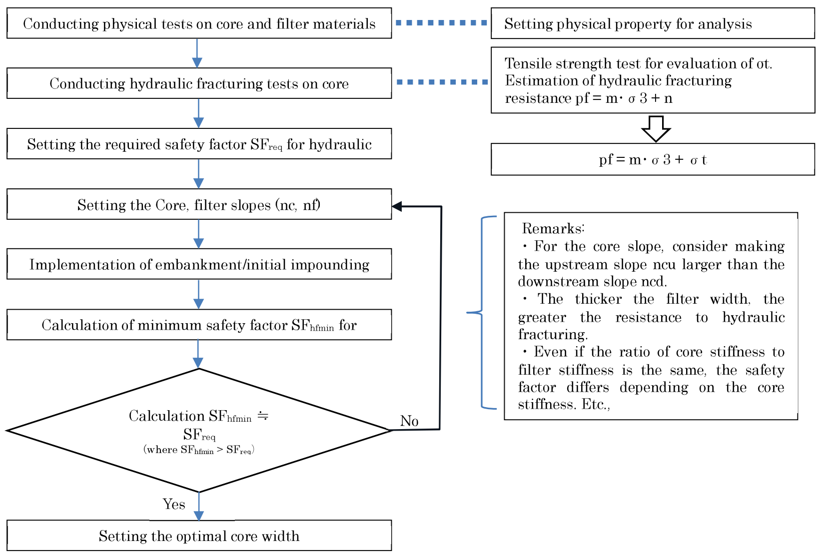
[1] Filters and leakage control in embankment dams, Sherard, J L; Dunnigan, L P
In: Seepage and Leakage from Dams and Impoundments (papers to the Symposium, Denver, Colorado, 5 May 1985)

to set the tensile strength σt, and then determine the hydraulic fracturing resistance in the form $pf = m \cdot \sigma 3 + \sigma t$.

2. Set the required safety factor SF_{req} for hydraulic fracturing to be used for evaluating the safety factor of hydraulic fracturing. It is required to have a discussion in setting SF_{req}. AT present SF_{req} may be set at 1.2 same as the slip safety factor, or at 1.0 with fracture progression analysis as necessary to evaluate the extent of the fracture area.
3. Next, after setting the initial values of "nc" and "nf", an embankment and impounding analysis will be conducted, and SFhf against hydraulic fracturing after impounding is calculated. Then seek "nc" and "nf" where SF_{hf} is closest to SF_{req}. These "nc" and "nf" will be the optimal core widths. The premises is that the safety factor be met with requirement using the current design method, slip stability analysis.

In setting the core slope (nc) and filter slope (ns) during the process of determining the optimal core width, consider the following findings/knowledge.

- Regarding the core gradient, if the upstream gradient ncu is made larger than the downstream gradient ncd, the resistance to hydraulic fracturing will be greater under the condition that ncu+ncd=constant.
- The greater the filter slope (the thicker the filter width), the greater the resistance to hydraulic fracturing.
- Regarding the combination of core stiffness and filter stiffness, even if the ratio of core stiffness to filter stiffness is the same, the safety factor differs depending on the magnitude of core stiffness.
- Even if the ratio of core stiffness to filter stiffness is the same, the safety factor differs depending on the core stiffness.



Flow of Rational Design Method for Core Width based on Hydraulic Fracturing Resistance

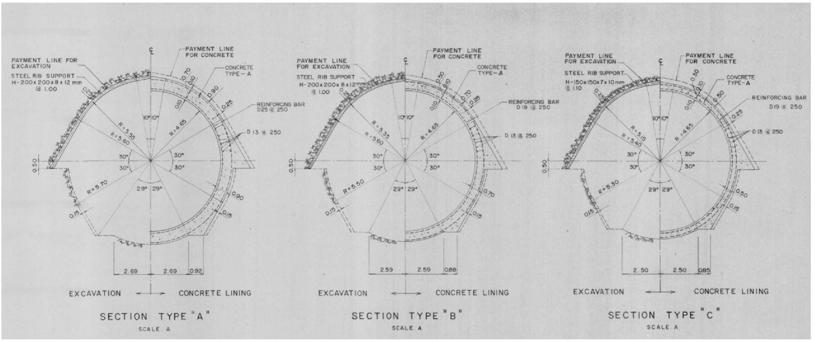
3. Conclusion

Since no hydraulic fracturing tests have been performed on core material at this time, it is recommended to take following steps minimizing the required amount of core material:

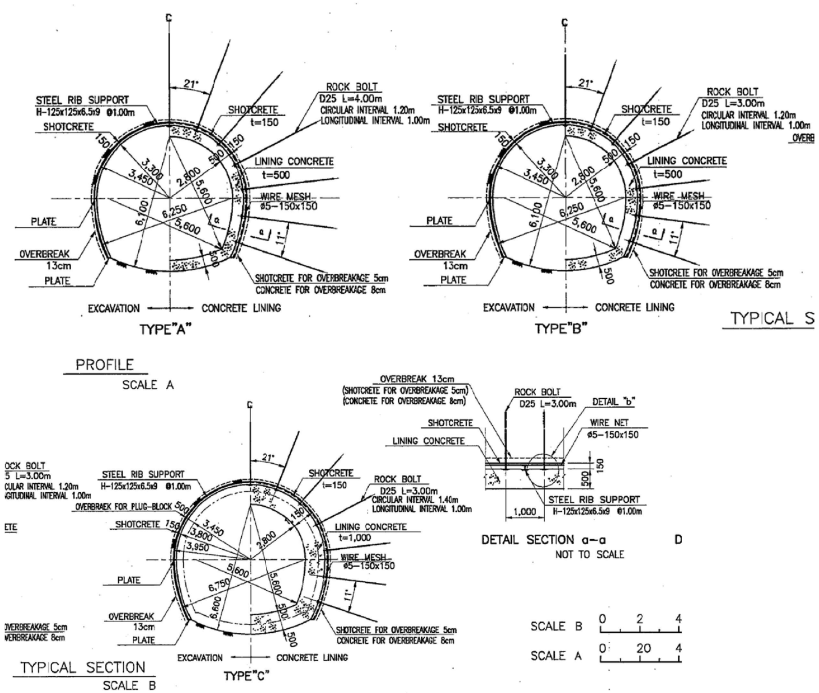
1. Determine the minimum width by considering the core top width and the performance of the heavy construction equipment used. In my experience, it is 4 meters. According to The EM US army, the minimum width is 10 feet.
2. At present, it is considered appropriate to set the upstream/downstream gradient of the core as 1:0.2. In some cases, a ratio of 1:0.16 to 0.17, which is slightly steeper than 1:0.20, has been adopted in combination with a filter that satisfies Sherard's limit filter criteria, which stipulates stricter piping conditions than current filter standards. If you apply this, it is necessary to check the herard's limit filter criteria and stability of dam.
3. Based on the seepage failure cases, the ratio of core width (B) to dam height (H) should be at least B/H = 0.25 or more.

Attachment 2

Payline for diversion Tunnel: Bili-Bili Dam



Payment for diversion tunnel: Jatibarang dam Project



4.1 *UNDERGROUND EXCAVATION (Specification of Jatibarang Dam)*

4.1.2 *Shape and Limits of Excavation*

a. All underground excavation shall be made to the lines, grades and dimensions shown on the Drawings or directed by the Engineer.

b. No unexcavated material of any kind will be permitted to remain within the lines shown on the drawings.

c. The overbreak lines indicated on the drawings are the limits to which payment will be made regardless of how far the limits of the actual excavation fall outside of the overbreak lines.

d. The nature of the materials being excavated could, for subsequent excavation, make it necessary, as determined by the Engineer, to move the payment line for excavation to provide for increased thickness of concrete. The Contractor shall not be entitled to any additional compensation because of such changes other than resulting from the increased quantities due to the new positions of the payment line for excavation.

e. Any and all over-excavation performed by the Contractor for any purpose or reason, except as may be directed, and due to the fault of the Contractor, shall be at the expense of the Contractor. Unless otherwise directed or approved by the Engineer, all such over-excavation shall be backfilled with concrete or shotcrete and the cost of furnishing and placing this backfill shall be at the expense of the Contractor.

f. For each advance in the tunnels the Contractor shall survey the actual excavated profile after scaling to ensure that the profile conforms with the minimum excavation requirements and to determine the extent of overbreak. The method of survey shall be as proposed by the Contractor and approved by the Engineer. Results of all such survey shall be submitted to the Engineer.

g. The Contractor shall not carry out local widening of any excavation for his own purposes without prior approval. Where such approval is granted it will be on the express condition that the resulting over-excavation shall be completely backfilled with concrete, in an approved manner, at the expense of the Contractor. Any required support shall also be at the expense of the Contractor.

4.1.8 *Measurement*

Measurement, for payment, of Underground Excavation for Diversion Tunnel and Underground Excavation for Outlet Tunnel (Items D 1.1 and D 1.2) shall be made of the in-situ volume of actual material excavated up to the limit of the overbreak line for excavation shown on the Drawings and will be made along the centreline of the excavation between the limits of excavation shown on the Drawings or directed by the Engineer.

4.1.9 Payment

a. The rates in the Bill of Quantities for Underground Excavation shall include all labour, equipment and materials required for excavation within the specified limits, removal and disposal of excavated materials, water control, ventilation, heading and benching, the development and testing excavation methods and all other work necessary for excavation in the tunnels as shown on the Drawings. These rates shall also include for excavation cycles times to suit the installation of support required in the various sections of each of the tunnels.

b. Payment for Underground Excavation for Diversion Tunnel (Items D 1.1) will be made at the rate per cubic meter tendered therefor in the priced Bill of Quantities

c. Payment for Underground Excavation for Outlet Tunnel (Items D 1.2) will be made at the rate per cubic meter tendered therefor in the priced Bill of Quantities

9.30.2 Measurement

c. Payment Lines for Diversion Tunnel Lining:

Measurement, for payment, shall be made to the neat lines of the inner surface and end faces and to the approved inner surface payment line for shotcrete. (Ref. Item I.9)

d. Payment Lines for Outlet Tunnel

Measurement, for payment, shall be made to the outer surface of the steel outlet pipe and the approved inner surface payment line for shotcrete. (Ref. Item I.12.2)

e. Payment Lines for Backfill Concrete

Measurement, for payment, for concrete required for backfill of seams, cavities etc. excavated in accordance with Sub-Clause 3.4.1.3 (Item I.13.2) and abandoned exploration adits (Item I.12.4) shall be made of the actual volume of concrete placed in these locations as directed.

Measurement of concrete placed in over-excavated areas as specified in Sub-Clause 3.4.1.1 para (i) shall not be measured for payment.

9.30.3 Payment

a. General

Payment for concrete in the various parts of the Works will be made for the respective volumes of concrete measured in accordance with Sub-Clause 9.30.2 at the rates per cubic meters tendered in the Bill of Quantities for the payment items for each type of concrete. The rates shall include the cost of all labour, equipment and materials required for the planning and testing, furnishing, mixing, transporting, placing and curing of concrete, furnishing and placing forms, supports, scaffolding and of the finishes applied to each item of concrete work.

Payment items to be paid under this Sub-Clause 9.30.3, paragraph a are : I.9, I.10.1, I.10.2, I.10.4, I.10.5, I.10.4, I.11, I.12.1, I.12.2, I.12.3, I.12.4, I.12.5, I.13.1, I.13.2.

Payment provisions, additional to those specified in this paragraph a, specific to particular payment items, are specified hereunder.

b. Concrete Type D in Concrete Plug in Diversion Tunnel

Payment for Concrete Type D in the concrete plug in the diversion tunnel (Item I.12.3) shall include the cost of cooling the concrete as specified in Clause 9.11.11 and the cost of injection grouting as specified in Clause 5.10.

Site Inspection Report

Hiroshi SHIMIZU
JICA Dam Construction Advisor

Name of Dam : Menintin Dam
Date : February 21, 2024

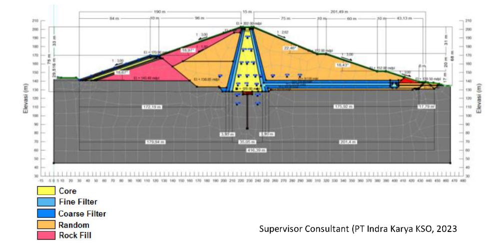
1. General

1.1 Meninting Dam

Although the western region of the Lombok River System has a relatively abundant amount of water potential, it is anticipated that shortage of water at irrigation areas in the central and southern regions. Topographically, it is possible to provide additional water from Meninting river to approximately 70,000 hectares of irrigation area via existing irrigation canals (HLD (High Level Diversion) Jangkok Babak and HLD Babak–Renggung–Rutus (Pandan Duri)) connect to the Lombok River system.

The Meninting dam was proposed, particularly, in response to the increasing of water demand in the western region of Lombok Island.

Dam Name	: Meninting
Province	: Nusa Tenggara Barat
Regency	: Lombok Barat
Coordinate	: 8°31'11" S 116°9'10" E



Main Dam	
Type	: Radom filled dam with central core
Height	: 74.0 m
Elevation of crest	: +202.0 m
Elevation base foundation	: +128.00 m
Wide of crest	: 15.0 m
Lenght of crest	: 430.0 m
Upstream slope	: 1:3.0
Downstream slope	: 1:2.6(top) & 1:3.0(bot)

Reservoir	
Elv. of normal water level (NWL)	: +196.0 m
Elv. of low water level (LWL)	: +169.0 m
Elv. of flood water level Q1000	: +198.80 m
Elv. of flood water level QPMF	: +200.60 m
Normal storage capacity	: 9.91 million m³
Dead storage capacity	: 1.72 million m³
Effective storage capacity	: 8.19 million m³

Hidrological Discharge	
5 year return period	: 169.68 m³/s
10 year return period	: 207.18 m³/s
25 year old return period	: 274.14 m³/s
50 year return period	: 238.6 m³/s
100 year return period	: 315.24 m³/s
1000 year return period	: 377.45 m³/s
PMF return period	: 830.49 m³/s

Intake	
Type	: Tower
Tower height	: 33 m
Intake inlet elevation	: +168.0 m
Gate dimension	: 6 x 6 m

Geology	
Formation	: Lekopiko
Rock/soil	: Breccia, andesite lava
Rock Class (CRIEPI)	: D, CL,CM
Main Dam Foundation	: Breccia, andesite lava
Spillway Foundation	: Breccia, andesite lava
Diversion Foundation	: Breccia

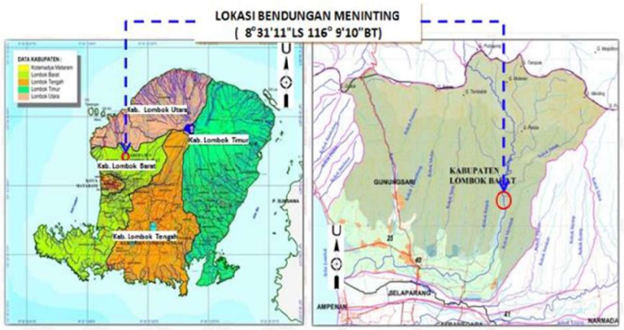
Spillway	
Type	: Side channel (tunnel)
Length	: 357.0 m
Wide threshold	: 30.0 m
Crest threshold elevation	: +196.0 m
Stilling Basin type	: Flip Bucket

Diversion	
Location	: Right abutment
Type	: conduit & tunnel
Length	: 118 m & 308 m
Dimensions	: 5.0 x 6.0 m
U/S elevation	: +148.57 m
D/S elevation	: +147.72 m

(1) Location and Purpose of Bagong Dam

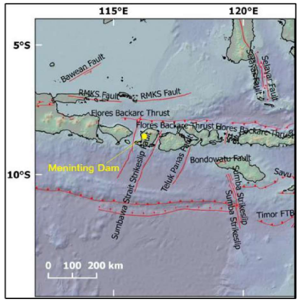
- ✓ Administrative Area : Bukit Tinggi Village, Gunung Sari District (Right Side of the Dam) and Gegerung Village & Dasan Griya Village, Lingsar District (Left Side of the Dam), West Lombok Regency
- ✓ Geographical Position : 8031'11" South Latitude; 11609'10" E
- ✓ Name of River : Meninting

- ✓ Name of River Basin : Meninting
- ✓ River System : Lombok



(2) Regional Geology

Dam foundation rock is formed with Lekopiko Formation (Qvl) consist of Pumiceous tuff, laharic breccia and lava.



Although in Lombok Island no active faults have been identified, the Lombok Island is surrounded by active faults in the sea. In the north, there is the Flores Backarch Thrust, in the west there is the Lombok Strait Fault and in the east there is the Sumbawa Strait Fault. In July-August 2018 there were several major (>6 M) earthquakes on the island of Lombok which caused a lot of damage and observed signs of liquefaction in several places.

These conditions make seismic activity on the island of Lombok quite intensive.

Based on the geological analysis during detailed design (2017), the foundation of the left dam abutment on volcanic breccia rocks with excavations 4 to 18 meters deep. At the riverbed, the river sediment depth is 26 meters deep to reach the fresh breccia. While on the right abutment terrace deposits and strong weathered breccia rocks is 5 to 10 m deep to reach the fresh breccia.

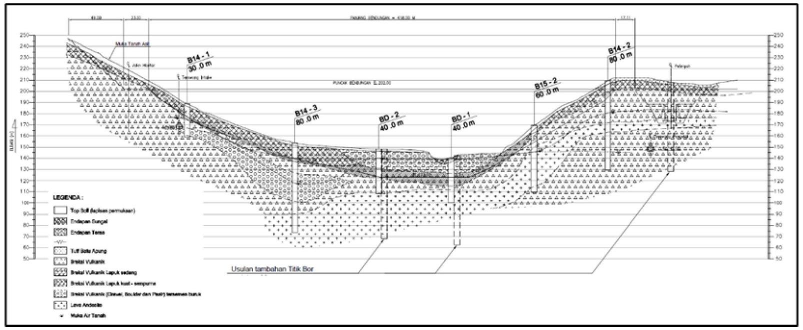
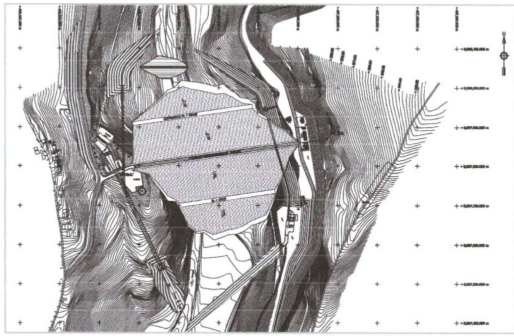


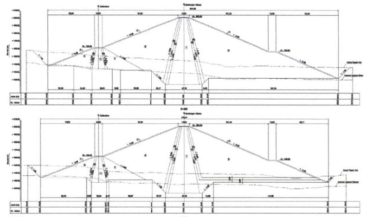
Figure 1 Engineering Geological Profile of Meninting Dam

The condition of the dam foundation based on laboratory testing of core samples of volcanic breccia rocks in 5 drill holes namely B4-3, B15-2, B15-4 and B15-7 in CL class rocks showed unconfined compressive strength values of 8 kg/cm2 and 19.26 kg/cm2, while CM class rocks show compressive strength of 60.63 kg/cm2 and 65.40 kg/cm2 and CM-CH class rocks have compressive strength of 108.70 kg/cm2, poisson ratio between 0.271 to 0.362, and young modulus between 1.7.49+02 to 9.50+04 kg/cm2.

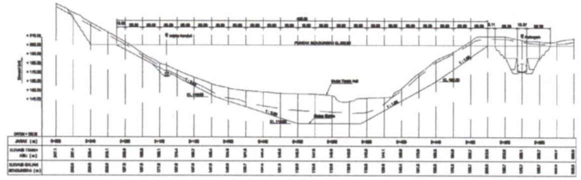
(3) Drawings of Plan-Profile-Section and Spillway Structure



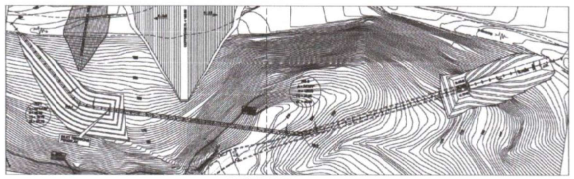
Gambar 2. Denah tata letak Bendungan Meninting dan bangunan pelengkap (desain)

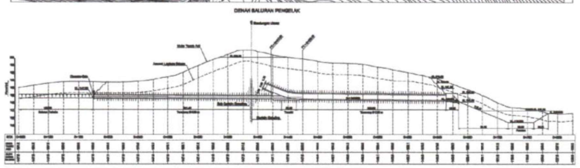


Gambar 4. Potongan melintang tubuh bendungan (desain)



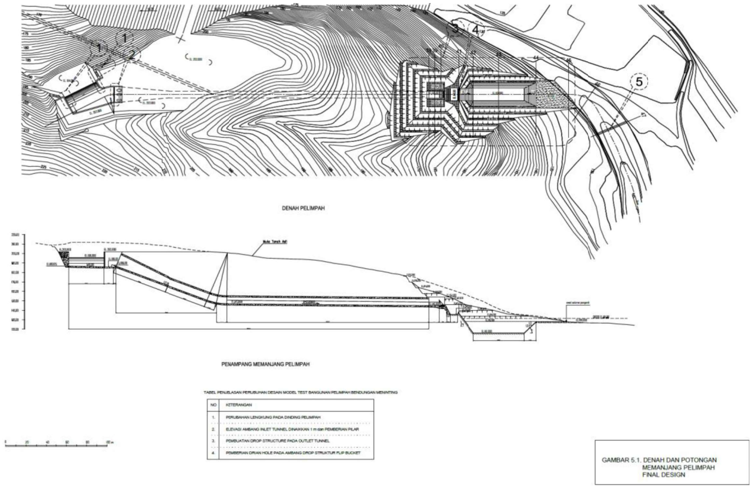
Gambar 3. Potongan memanjang tubuh bendungan (desain)





Gambar 5. Denah dan potongan memanjang saluran pengelak (desain)

Diversion Tunnel



Spillway Structure

2. Progress/Schedule as of February 2024

At the time of the inspection, dam embankment works has been started.

The physical progress of the Meninitng Dam as of February 21, 2024 is 75.4%. Impounding target is planned in May 2024.

Dam embankment work on Meninting Dam

3. Issues Discussed

3.1 Discussed Points

(1) Random Material

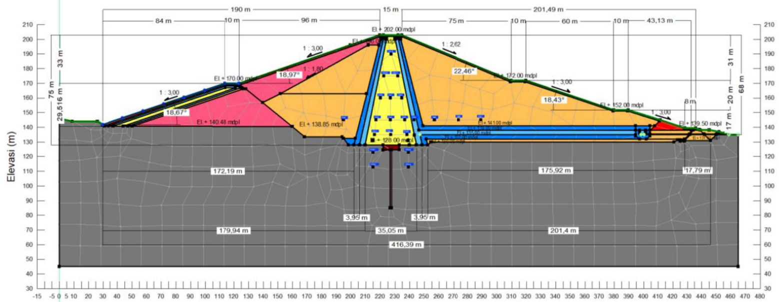
The embankment material for random zone contains much clay materials, and clay clumps were observed in the spread material at the embankment surface. This character of random material indicates that the material are not uniform and low permeability.

It is necessary to establish quality control method of random materials.

(2) Filter between Rock, Random & Core Materials

In the random zone, clay components are richly contained and it is almost the same condition as the core material. Filter is provided between core and random zone. However, there is no filter zone between rock zone composed with coarse material and random zone.

It is recommended to check the necessity of the filter between random and that core and the filter between random and rock zones. Dam stability shall also be rechecked applying the new seepage line.



(3) Dry Condition of Embankment Surface

It was observed that that the core embankment surface is too dry. Prior to commence the next layer embankment of core material, water content adjustment and raking of embankment surface should be conducted.

Note : The day after I returned to Jakarta, the Contractor sent me photos of the treatment of embankment surface of core. Judging from the photos, it appears that the process is being carried out properly.


Raking of Embankment Surface


Watering


Condition of after Watering


Spreading Core Material

(4) Land slide at Borrow Pit

Sliding has occurred at the excavation slope of borrow pits.

The development plan of borrow pit was not established and the Contractor collecting materials from the toe of the area. The excavation surface is steep and there is a high risk of collapse.

To avoid further risks, the Contractor should immediately draw up an excavation plan for the Borrow Pit, including the excavation procedure, and proceed the work in accordance with this plan.

(5) Grouting

Since this kind of soft rock in Meninting dam foundation (Pumiceous tuff and laharic breccia), in general, it is supposed that grouting effect in foundation rock has less wide penetration. It is recommended to review all grouting data and to ensure that the grouting is in accordance with the specification again.

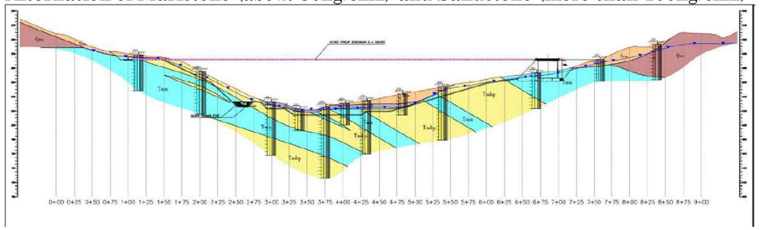
Suggestion on Grouting Works, Cijurey Dam

JICA Dam Construction Advisors
Hiroshi Shimizu
Naoya Mizuno

1. Foundation

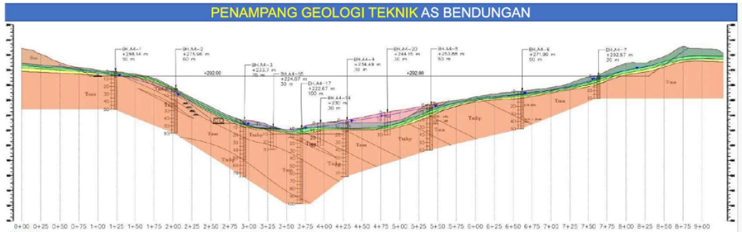
(1) Foundation

i. Alternation of Marlstone (about 30kg/cm2) and Sandstone (more than 100kg/cm2)

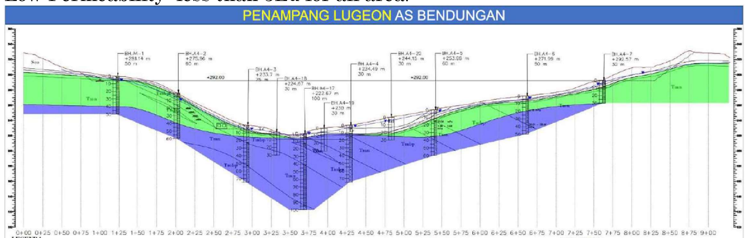


ii. Axis of anticline runs upstream to downstream.

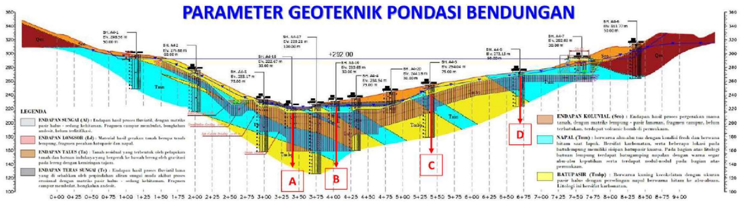
iii. Classified as CM class Rock



iv. Low Permeability: less than 5Lu for all area.



But it is assumed that some part of foundation rock contains high permeability zone (need to confirm).

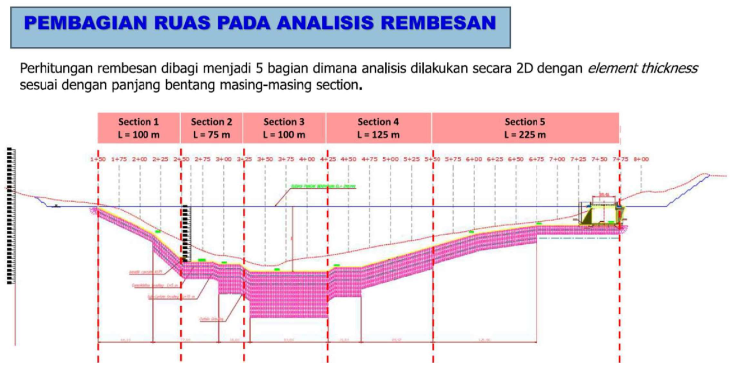


Parameter	Titik A	Titik B	Titik C	Titik D
Titik Bor	BH.A4-18	BH.A4-19	BH.A4-5	BH.A4-6
El. tumpuan bendungan	+215 mdpl	+215 mdpl	+242.05 mdpl	+268.23 mdpl
Kedalaman titik bor	9.57 m	14.84 m	11.84 m	3.72 m
Litologi	Batupasir	Batulempung napalan	Batupasir	Batulempung napalan
Rock Class	CM	CM	CL	CL
Core recovery	100%	100%	100%	100%
R.Q.D	75%	84%	10%	30%
N-SPT	>100	>60	>50	>50
Permeabilitas	1.62 × 10[illegible] cm/dt	8.77 × 10[illegible] cm/dt	1.51 × 10[illegible] cm/dt	6.04 × 10[illegible] cm/dt
Lugeon	15.43	0.83	14.38	6.96
Unconfined Compression Strength (UCS)				
Modulus elastisitas bebas (Eaxial)			1.35 E+04 kg/cm²	1.92 E+03 kg/cm²
Kuat tekan bebas (σC)			204.972 kg/cm²	28.371 kg/cm²
qu	28.16 kg/cm²	21.06 kg/cm²		
Axial strain	2.84 %	3.11 %		
Triaxial Test				
Kohesi (C)	3.36kg/cm²	3.06kg/cm²		
Int. Friction Angel (φ)	21° 30′	21° 19′		

(2) Design

Area of Curtain Grouting is determined based on the result of seepage analysis (?).

Rim grout is not considered (there is no geological survey at both rims).



2. Suggestion

(1) General

Since Geology is not uniform and contains many unknown factors, grouting works has been conducting in standard procedure, in general.

In the Cijurey damsite, Marlstone has a hardness of around 30 kg/cm2, and sandstone has a hardness of over 100 kg/cm2, so cracks may be formed in sandstone due to folding (similar to Juragung Dam). It is supposed that cracks do not develop much in marlstone (soft rock).

Furthermore, since the strata (folds) run in the upstream to downstream, cracks tend to develop aling the same direction in the foundation rock, and this type of crack is likely to trigger dam failure due to piping at the dam foundation.

Therefore, high attentions will be required when planning grouting works and during construction. In particular, when conducting the grouting works in Marlstone, which is a soft rock, special consideration will be required to the injection pressure, thickness of cement milk and distance of grouting holes.

(2) Area of Curtain Grout

The area of curtain grout is determined based on the Lugeon map (permeability of foundation). If the Lugeon value is as per the existing Lugeon map (Lu is uniformly less than 3), the injection depth will be set according to the water pressure, so sudden changes of grouting depth like the current design will not occur. We recommend checking the detailed Lugion Value and resetting the range.

Furthermore, there is no plan for curtain grouting on both rim sections at present. Since a geological survey has not been conducted at these sections, the permeability of the actual ground has not been confirmed. So a geological survey of at both rim sections and a rim grouting plan should be carried out.

(3) Grouting Plan

Prior to the grouting works, Grouting Test shall be carried out to obtain the character of foundation rock and to determine the following specifications of grouting works;

- maximum pressure
- changing criteria of thickness of cement milk (Water-Cement ratio) during injection
- intervals of specified holes

Inspection Report
Bagong Dam

JICA Dam Construction Advisors
Hiroshi Shimizu

Day / Date	: Thursday, 2nd May 2024
Location	: Bagong Dam
Time	: 10.00 WIB – Finished
Event	: Site Inspection of Bagong Dam Construction

1. General

The construction of Bagong Dam is targeted to be completed at 2024. Callently, the progress of Bagong Dam project is 46%. The main issue of Bagong is treatment method of colluvium layer at right (spillway foundation) and left dam foundations. The diversion works of Bagong dam had been completed and foundation excavation and grouting test of foundation have been conducted. The construction of spillway was not started yet (under progress of pile installation).

2. Field Inspection and Discussion

There are 3 alternatives for colluvium foundation treatment that had been proposed by consultants for construction supervision:

(1) Maintaining the dam axis as original design (667 m in length) and removing/excavating entire existing colluvium on foundation up to the bedrock. The depth of excavation may be between 20-30 m from the original ground surface.

(2) Maintaining the Bagong dam axis as original design (length 667 m), and excavating colluvium partially as submitted to BTB. Secant piles will be provided in the remaining colluvium along dam axis (Tugu Dam for reference).

(3) Modification of Dam Axis: The dam axis on the left abutment is shifted to upstream to get the thin colluvium (length of dam axis to be 710m). Based on two (2) additional core drilling results, depth of the colluvium is around 27 m.

3. Comments

(1) Alternative 1 is the safest condition (accepted), but constrained with financial issue. Alternative 2 might not be applied to Bagong dam due to difference condition of colluvium condition between Bagong dam and Tugu dam and secant pile cannot be used as reinforcement. The colluvium condition of Tugu Dam is coarse grained colluvium that settlement is not occurred (in Bagong Dam has settlement potential). Alternative 3 may be applied but take more time and not economical.

(2) The best condition of dam foundation is all of dam body lie on good bedrock foundation. For Bagong dam, it is possible that "Core" and "Filter" materials should be placed on bed rock and remaining colluvium in the dam body may be remained as tolerated, as shown Figure 1 (should be confirmed by stability analysis).

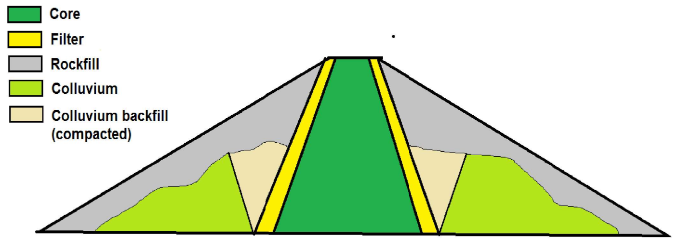


Figure 1 Sketch of Bagong dam design with remain colluvium deposit

(3) The colluvium will be regarded to be a part of dam body in all cases. Therefore, it is anticipated to ne required several countermeasures such as counterweight based on the results of stability analysis and the several facilities such as inspection road, instruments might be relocated.

(4) at the right abutment near the spillway, the dam foundation is still on colluvium. It recommended that all colluvium deposits should be removed and replaced with dental concrete or all replaced with core fill material completely (removing the installed secant piles as well).

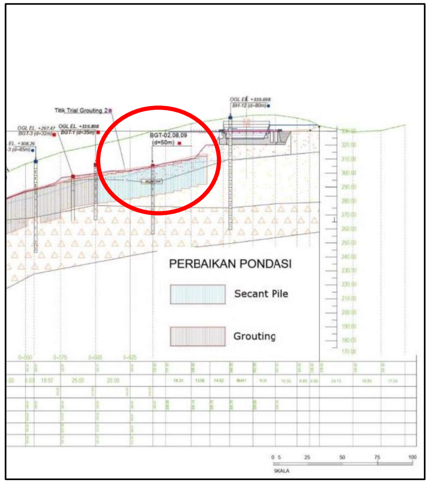


Figure 2 Colluvium at right abutment foundation

(5) In case the secant pile employs, it is necessary to pay attention to the connection between the secant piles and the core foundation. In addition, it is necessary a

Discussion on Dam Foundation, Jenelat Dam

JICA Dam Construction Advisors
Hiroshi Shimizu

1. Jenelata Dam General Information

1.1 Location:

Jenelata Dam is located near Pattalikang Village, Manuju District, Gowa Regency, South Sulawesi Province. The location of the dam is on the Jenelata River and the upstream of the confluence of the Jeneberang River and Jenelata River. This project is located between 5° 15′ -5° 20 ‘S and 119° 36 ‘-119° 40 ‘E.

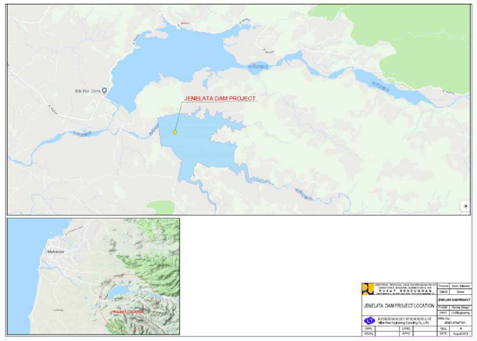


Figure 1.1 Jenelata Dam Location Map

1.2 TECHNICAL DATA

Technical data of the Jenelata Dam is as follows:

Reservoirs

	Amount	Unit
Watershed area	222.62	Km ^{2}
Inundation Area	12.20	Km ^{2}
Reservoir Max Storage	223.60	Million m ^{3}
Storage Effective Reservoir	174.50	Million m ^{3}

Body Dam

Type Dam	: *Concrete Faced Rockfill Dam* (CFRD)
Dam Height	: 62.80 m
Dam Length	: 1,525 m (Left = 713 m, Right+ 691 m, +121m)
Crest Width	: 10.00 m
Crest Elevation	: +105.80 masl
Slope	: Upstream 1:1.4 Downstream 1: 1.5

Overflow

Type	: Middle Spillway
Spillway Elevation	: + 99.50 masl
Base Elevation	: + 42.50 masl
Width of Overflow + Diverter + Waster	: +121.00 masl

Cofferdam

Elevation *Cofferdam*	: + 64.00 masl
Cofferdam Water Level Elevation	: +62.50 masl
Peak Width *Cofferdam*	: + 6.00 m
Cofferdam Length	: + 518 m (Weir right) + 280 m (weir left)

Diversion Channel

Channel Base Elevation	: +55.00 to +55.50 masl
Depth *Channels*	: +10.00 masl
Channel Width	: +60.00 masl
Diversion Channel Length	: 416 m

JENERD0201-A Brief Report January 2020

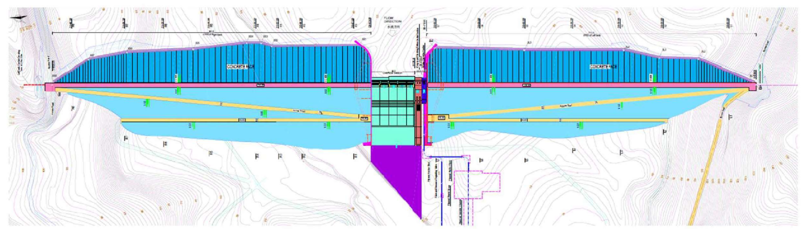


Figure 6-1 General Layout of Complex Works of Combination of CFRD Sections and Concrete Gravity Dam Section

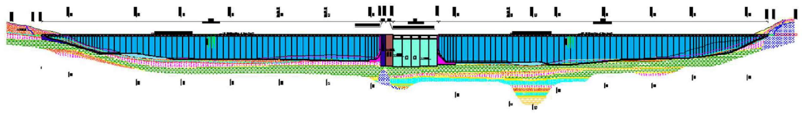


Figure 6-2 Upstream Elevation View of Combination of CFRD Sections and Concrete Gravity Dam Section

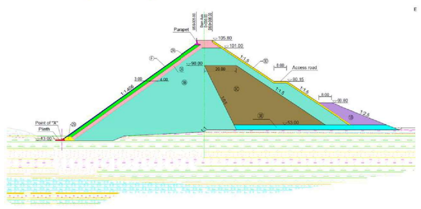


Figure 6-3 Typical Profile of Concrete Face Rockfill Dam

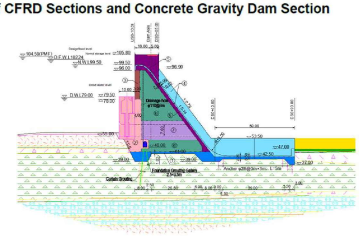
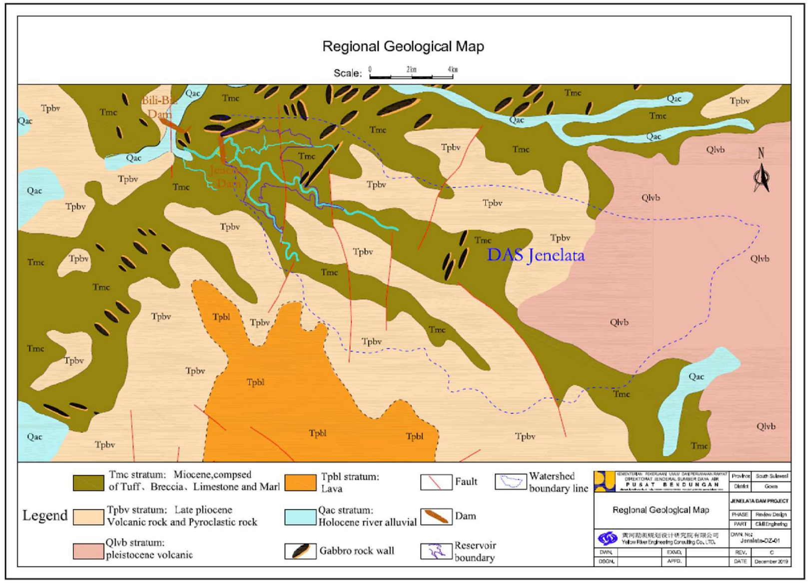


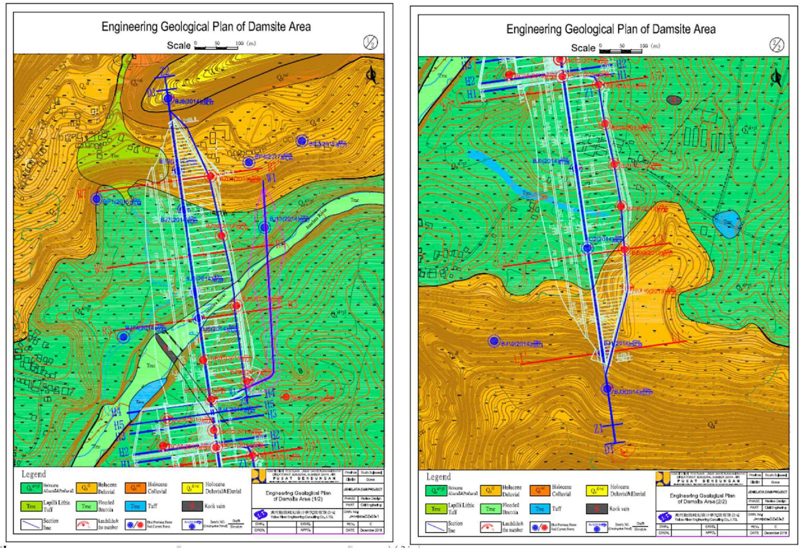
Figure 6-4 Typical Profile of Overflow Section (1 # - 3 # sub-sections)

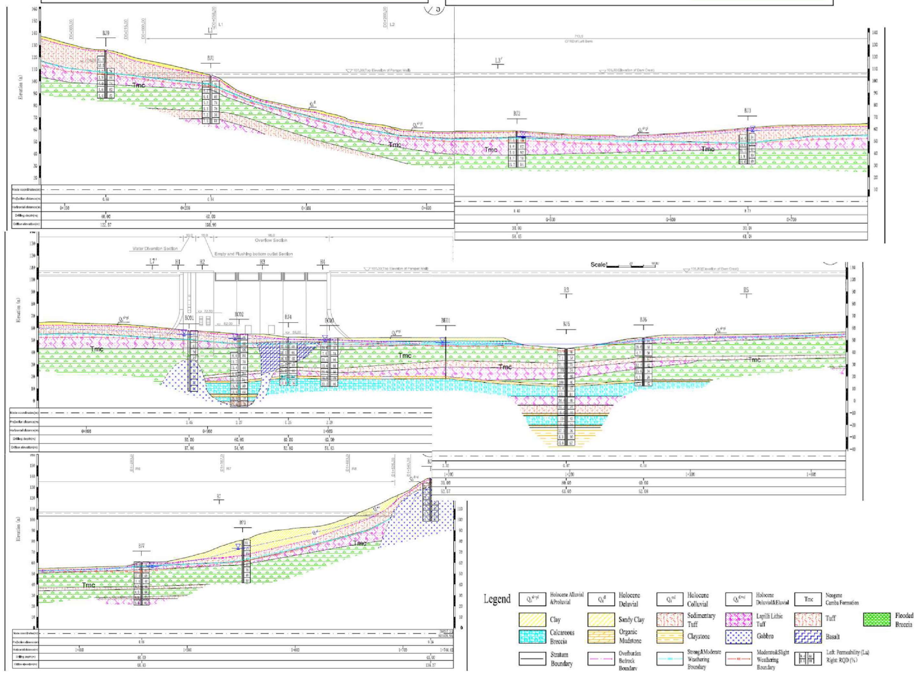
2. Geology

2.1 Reservoir Area

The strata exposed in the reservoir area include: Miocene to Pliocene pyroclastic rock, lava rocks and breccia (Tmc), Miocene to Pliocene intrusive rock, and Quaternary loose accumulation layer (Q4).







2.2 Damsite

Jenelata Dam site area is located in a valley with open topography. The difference of topographic relief in riverbed and terrace is small. The slope of both left and right abutments is generally gentle, which is 15 20°, and is locally steep. The thickness of the overburden of the river bed along the dam axis is generally around 1.0m the bedrocks are buried shallowly, and exposed in the riverbed. The underlying bedrocks are mainly tufa, lapilli tufa, and diluvial breccia, etc.

The top of bedrocks in the valley is generally distributed with completely weathered and heavily weathered layers with the thickness of 5.0 to 9.0m, and the weathered layers at the abutments of both banks are thicker.

3. Discussion

The site inspection was conducted on 30 July 2024.

At the time of the inspection, the construction progress had reached 3.833%. Ongoing work includes excavation of the main dam foundation, access roads for facility buildings, etc. Based on the results of the inspection and discussions in the field, the following are recommended as follows:

The main discussion in the meeting were as follows, and I proposed responses to them.

Comment/Answer/Suggestion in the Meeting

	Comments by DSC	Answer by the Consultant	Advise by Dam Advisor
Q1	What are you going to conduct a slaking protection measure on excavated surface of tuff and claystone that are prone to slaking (they start slaking in a few days)?	Please advise us appropriate countermeasures.	⇒ leaving about 50 cm of cover rock and protecting the excavation surface with shotcrete, and so on. These measures had been provided in BiliBili dam construction.
Q2	The prince foundation includes tuff breccia and the boundary between tuff and mudstone strata with different strengths and remaining colluvial at some parts. Have you analyzed the deformation?	In China, it is generally applicable this kind of foundation as an prince foundation.	⇒ Check the deformation module and future deformation amount with a loading test.
Q3	How did you decide on the tuff breccia design value for the gravity dam foundation? Have you conducted a block shear test?	The design parameter has determined applying laboratory tests results. Insitu direct block shearing test is not conducted.	⇒ Insitu direct shearing test can be performed without an audit, so it shall be conducted and re-confirm the validity of applied design parameters.
Q4	The dam is zoned applying a rock zone (basalt) for the upstream part of the main body and random zone (tuff breccia) for the downstream. Tuff breccia, however, is prone to fine graining after compaction. Please check the crushing condition, permeability, etc. with a trial embankment?	The Contractor will carry it out before commencement of embankment.	⇒ Conduct a trial embankment as early as possible. Use the construction equipment used on-site.
Q5	Single row curtain grout is applied in the design even in the soft rock area. What is the basis for a single row?	In China, single raw is generally applied considering economic advantage.	⇒ The number of curtain grout rows is determined based on the rock characteristics such as maximum injection pressure or critical pressure of rock, amount of cement milk that can be injected, and distribution of cracks in the foundation rock. In particular, in the case of soft rock, cement milk can not penetrate easily, so it is taken a method to ensure the water-stopping effect by a thicker curtain width. From a safety perspective, it may be better to install two rows. Please check with a grout test.

Comments on
Towards Initial Impounding of Lausimeme Dam

JICA Dam Construction Advisor

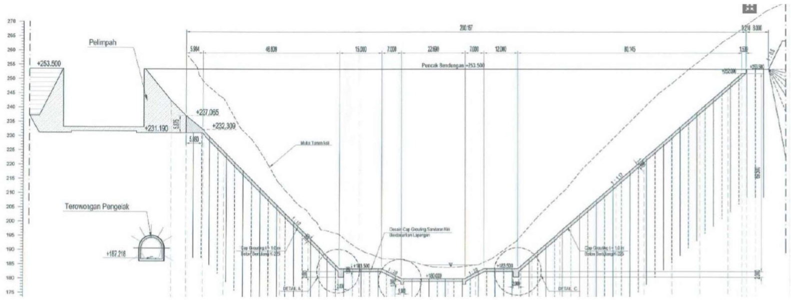
SHIMIZU Hiroshi

1. Lausimeme Dam

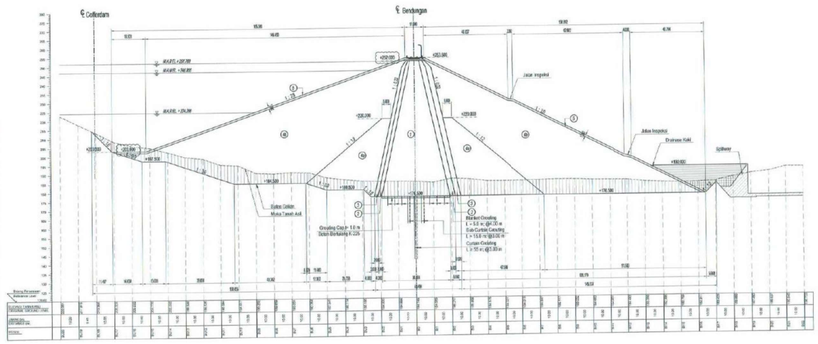
1.1 Location

1.2 Technical Data

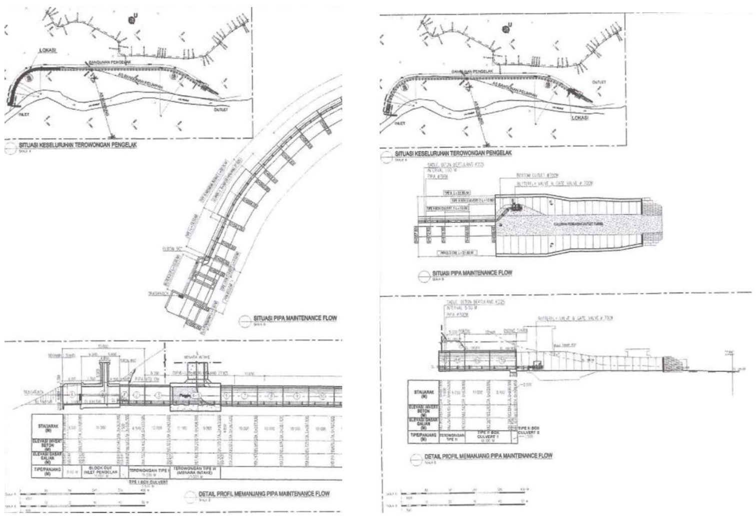
Layout Plan



Dam Profile along Dam Axis



Dam Section



Outlet Facility

1.3 Remaining Works

- The dam embankment has been completed up to the dam crest.
- All that remain works are adjacent works at the crest (road paving, etc.).
- The Contractor already ready for commencement of plug works.
- After closing the closure gate of diversion, the plug concrete works and the hydromechanical works for outlet facility will be started.

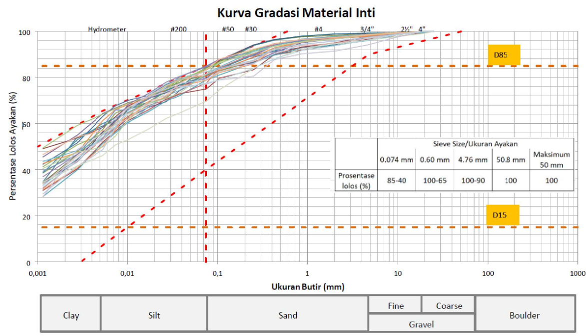
2. Issues Related to Impounding

The issues of concern regarding the commencement of the plug works are as follows:

(1) Settlement of Core Zone

The core material test results shown in the meeting showed that the grain size of core material was 80% of silt/clay (0.074mm or less), and 50% of clay (0.002mm or less). In addition, the consistency (Atterberg) test results show a maximum LL of 120, and some PL were over 30, so I understood that the uniformity of core material was not ensured (I'm not sure about the exact numbers, so need to check). Also, looking at the consistency, the LL are relatively large, so a considerable amount of settlement is expected. Furthermore, because of the rapid construction of core (dam of 73.5m high was constructed in one year), settlement of core did not progress sufficiently during the construction period, and there are concerns about large settlement after completion.

Generally, most of the amount of core settlement occurred during construction period, and the settlement should show a maximum value (several meters) near the middle elevation of the dam at the time of completion of the embankment. However, the observation results presented this time were only 40 cm, and it is assumed that the core did not settle during the construction period sufficiently.

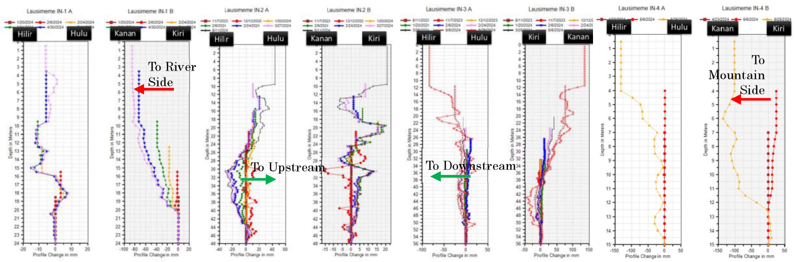


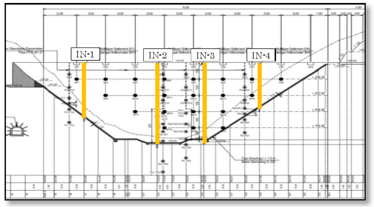
It is also assumed that differential settlement is occurring due to the non-homogeneity of the core material.

In general, the displacement of inside of core zone for both abutments inclines toward the center of the dam where settlement is the largest, and the displacements to the upstream and downstream sides are small.

However, there is displacement to upstream and downstream in Lausimeme Dam, with 4cm at IN-2 to upstream and 8cm at IN-3 to downstream in the opposite direction. Additionally, IN-4, installed on the right bank abutment, is inclined towards the mountain side. One possible cause is differential settlement, but the reason is unclear.

Since soil materials such as core materials have the ability to self-recover, it is thought that such problems will be resolved once consolidation is complete,



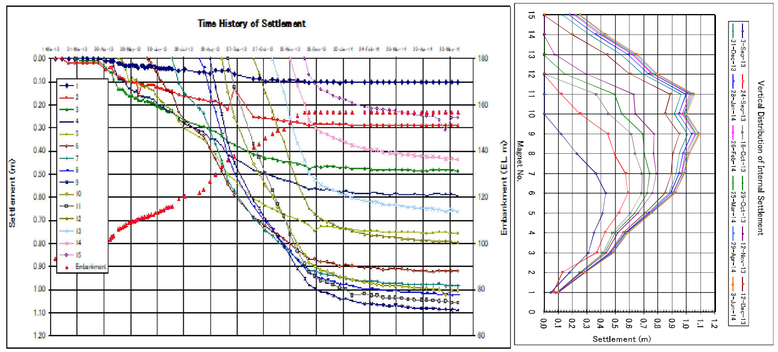


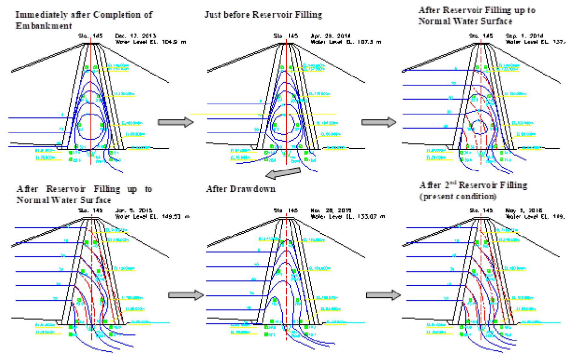
Monitoring Data of Inclinometer

Furthermore, due to the rapid construction, it is expected that there will still be a considerable amount of residual pore water pressure in the core. Before commencement of impounding, it is necessary to check the distribution of pore water pressure and the magnitude of excess pore water pressure.

Before impounding begins, it is necessary to re-check the amount of settlement and the distribution of pore water pressure to ensure safety of dam.

An example of how to summarize the data (Jatibarang Dam) is shown below for reference.





Before Impounding and 1 month after, 3 months after and 6 months after impounding

Fig. 4.1.11 Distribution of Pore Pressure

Example of Data Compilation for Confirming Amount of Settlement and Residual Pore Water Pressure (Jatibarang Dam)

(2) Pyroclastics in the Reservoir Area

The pyroclastics that spread throughout the reservoir area are made of glassy/angular sand with high permeability. Therefore, it is assumed that there is little risk of collapse or landslides due to a sudden rise in the reservoir water level. However, the pyroclastics are easily eroded by sudden changes in water level, in particular draw down of water level and waves, and there is concern that small-scale erosion could lead to wider collapse.

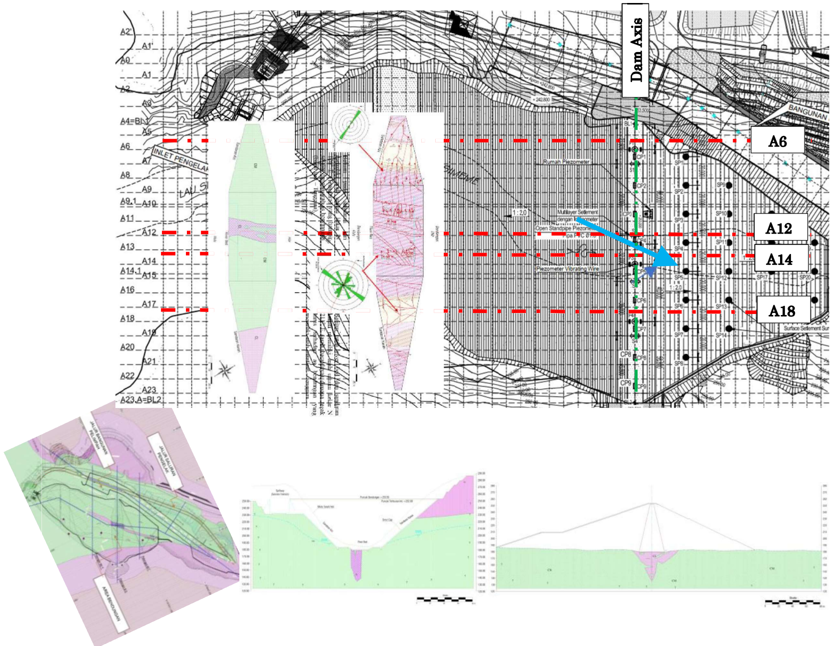
Before impounding, the condition of the pyroclastics in the reservoir area, especially above the low water level, should be inspected, and any areas that may lead to a collapse, should be removed. In addition, in areas where there is a risk of erosion by sudden water level change, waves and rainwater, it is necessary to install protection works to prevent erosion.

(3) Weak Zone in Dam Foundation

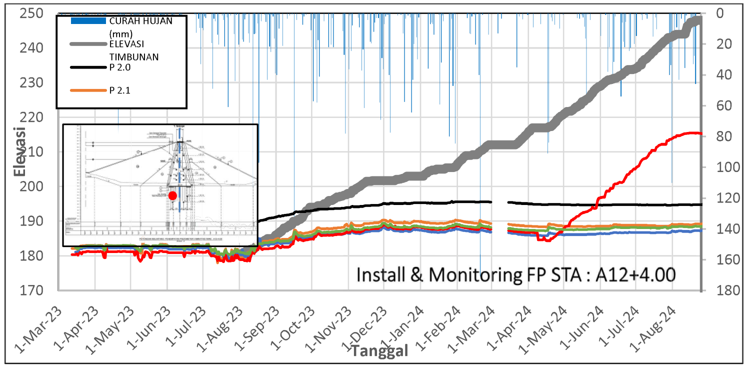
Among the piezometers installed in the dam foundation, pore water pressure has been rising suddenly since May 2024 at P2.3 (upstream of dam axis) on A12+400 section and P3.4 (downstream of the dam axis) on A14 section. And pore water pressures of only these two piezometers increase in accordance with embankment height. The installed locations of these piezometers are considered to be in a weak zone (CL class). No such phenomenon, however, has observed at the other two sections (A6 and A18).

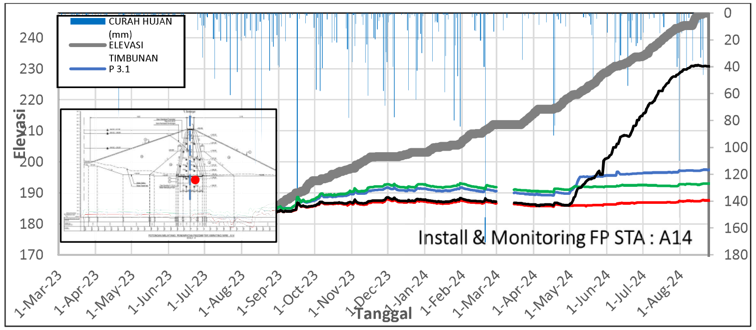
The information from these piezometers suggests that part of the foundation rock in the weak zone, connecting upstream and downstream of the dam, may have collapsed due to the extra load of the embankment. Judging from the proportional increase of pore water pressures to the load of the embankment, it is estimated that the collapsed rock is now directly affected by the overburden load of embankment. Furthermore, since the collapsed bedrock is expected to have deformed, there is a possibility that there are cracks that connect to upstream and downstream in the cover concrete (grout cap).

Before impounding the reservoir, it is necessary to clarify the cause of sudden change of pore water pressure and consider measures to be taken in the event of excessive leakage after impounding reservoir.



Distribution of Weak Zone in Core Foundation





Monitoring Record of Piezometers at STA A12+400 & STA A14

3. Recommendation

Towards initial impounding, firstly, issues regarding the safety of the Lausimeme dam, namely the progress of core settlement, the distribution and magnitude of residual pore water pressure, the erosion of pyroclastics, and singular values on the piezometer in the weak zone of the core foundation, must be clarified and any countermeasures that can be taken must be addressed, while emergency countermeasures must also be considered.

Also, since the outlet work will be installed after closing diversion tunnel, the water level cannot be lowered during the construction of the hydromechanical devices. This must be taken into consideration when planning countermeasures.

Monitoring plan of dam behavior also important and should be established prior to commencement of impounding,

Please thoroughly investigate, consider and settle each issue and then commence the initial impoundment.

Discussion on Tamblang Dam

JICA Dam Construction Advisors
Hiroshi Shimizu
Naoya Mizuno

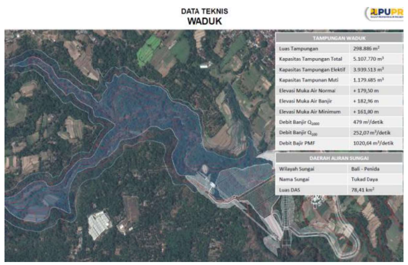
1. Tamblang Dam General Information

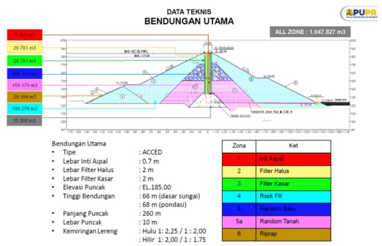
1.1 Location:

LOKASI BENDUNGAN TAMBLANG DI KABUPATEN BULELENG	
SANDARAN KANAN	DESA BILA KECAMATAN KUBUTAMBAHAN
SANDARAN KIRI	DESA SAWAN KECAMATAN SAWAN
GENANGAN	DESA BILA DAN DESA BONTIHING KECAMATAN KUBUTAMBAHAN DESA SAWAN DAN DESA BEBETIN KECAMATAN SAWAN

Figure 1.1 Sidan Dam Location Map

1.2 TECHNICAL DATA

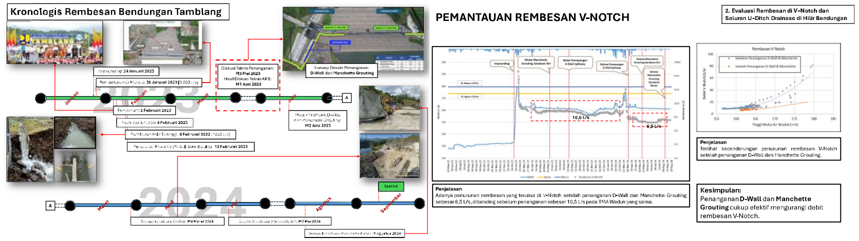




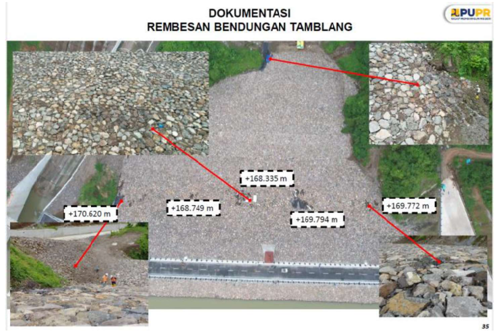
Reservoir

2. Leakage from Dam Body

2.1 Chronology of Leakage from Dam



Chronology and Observed Seepage from Dam (V-Notch)



Location of Leakage from Dam Body

2.2 Observed Phenomena of Leakage

The following phenomena were revealed by the information currently available.

✧ When the reservoir was impounded up to the normal water level (approximately EL.180m), 80ltr/sec of seepage water was recorded at the V-notch. Beside, at 5 locations on the downstream slope of the main dam, leakage were observed. The leakage point were in line at an elevation of EL.170m. Furthermore, leakage at the downstream toe of the left bank slope was found.

✧ From the aerial photograph, it seems that there is more leakage in the center.

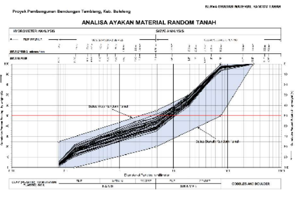
✧ During the on-site inspection in July 2024, traces of repaired cracks were confirmed on the pavement surface of the crest road parallel to the dam axis. The cracks occurred at both sides of dam (on the abutment slope), but not in the center. The cracks also occurred away from the center line, not in the center of the road.

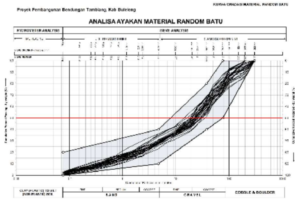
✧ It is possible that there was uneven settlement between the asphalt core/filter material and dam embankment. The monitoring records shows, displacement at along the dam axis is smaller than up & down stream sides of the crest road (dam axis: 80mm, Upstream: 140mm, Downstream: 140mm, on the section 05+10.55).

2.3 Considerations

(1) Location of Leakage on Downstream Slope

- The elevation of observed leakage corresponds to that of top of Zone 5 (Random Rock Zone, EL.170m).





- However, since the results of the field test during construction showed that the silt clay content of both Zone 4 (Rock Zone) and Zone 5 was 5% or less and expected coefficient of permeability of Zone 5 and Zone 4 is about 10-3m/sec, it is supposed that the leakage water would not reach the downstream side unless there is a significant leakage.
- Considering the embankment material, it is unlikely that the water would reach to the center of the dam, if it was only leaking from the abutment.
- It is supposed that this fact suggests the possibility of leakage from somewhere other than the abutment.

(2) Observation Results of Piezometer

- All piezometers installed in the filters on the three sections are linked to the water level and show almost the same value as the water level. In addition, the recorded values are almost the same upstream and downstream of the core.
- This suggests that the asphalt core is not functioning properly as a water stopper.
- The piezometers (PE-7 (upstream) and PE-8 (downstream) installed on the center of dam) have been measured immediately after the start of impounding and the results show that the upstream PE-7 rises in tandem with the water level, while the downstream PE-8 starts to rise slightly later. Since the impounding speed was extremely fast, it cannot be read accurately from the given graph at present, but an accurate analysis of this phenomenon may be helpful in estimating the leakage elevation.
- It is necessary to conduct a detailed analysis of the relationship between the monitoring records of PE-7 and PE-8 from the start of impoundment to the highest water level, the water level, and the amount of leakage.

(3) Observation Results of Inclinometer

- The inclinometer readings suddenly changed after a certain date. It is unclear whether this is due to measurement error or some external force. Needs confirmation.

(4) Construction of Asphalt Core

- Asphalt is placed with a high temperature of over 100 degrees Celsius and shrinks as it cools down to the surrounding temperature over time. It is known that the settlement of asphalt core is generally larger than filter.
- Due to the differential settlement of core and filter, the contact surface with the filter materials will be constrained then shearing stress occur at the

contacts. As a result, arch action occurs in the asphalt core, and tensile forces act, raising concerns about the occurrence of cracks.
- For this reason, it is ideal to install thermometers and strain gauges to check the actual condition as construction proceeds, but these devices were not installed in this dam.
- In case the dam embankment is rapidly constructed, the next layer will be placed in an state of expanded before cooling, so the stress generated will be even greater.
- If possible, it is recommended to estimate the expected shurinkage and check the differ between asphalt and filter based on the actual asphalt core construction.

(5) Geoelectric Survey Result

- The geoelectric survey results has been evaluated that the high conductivity area in the Section B (left bank ground) is connected to the high conductivity area that appears on the dam axis (Section C and D). But it is possible that the high condac The geoelectric survey results has been evaluated that the high conductivity area in the Section B (left bank ground) is connected to the high conductivity area that appears on the dam axis (Section C and D). But it is possible that the high conductivity area that appears on the dam axis is influenced by the spillway structure.
- Does the evaluation of the results take this into account?

(6) Others

- There is no measurement result of the amount of leakage during the construction stage (before impounding) to confirm the base flow.
- It is necessary to clarify the relationship between the reservoir water level and the groundwater level obtained from monitoring wells installed around the dam. Information before impoundment is particularly important for comparison.

3. Recommendation (Toward to Re-Impounding)

As a result of the completion of the construction of the diaphragm wall and Manshette grouting, a decrease in the amount of water leakage was observed but this was with the water level has been drawn down, and it is necessary to confirm the leakage condition when the water level rises (as Trial Re-filling).

However, prior to re-impounding, it is considered important to comprehensively compile and evaluate the monitoring data during construction, during impounding and after drawing down and re-confirm the cause of leakage and rout of leakage water to the dam downstream slope.

The items of the required investigation are shown below.

3.1 Compiling of Data

In this paper on our comments, only given information in the meetings was used, but in this materials, detail information of data during the construction stage and the process of estimating the cause of leakage has not been found.

As discussed above section, since there is possibility that water is leaking from the asphalt core, it is necessary to conduct the follows:

(1) to reevaluate the detail piezometer monitoring data with a focus on the characteristics of the asphalt core dam,

(2) to re-investigate the leakage points of dam and its surroundings, and

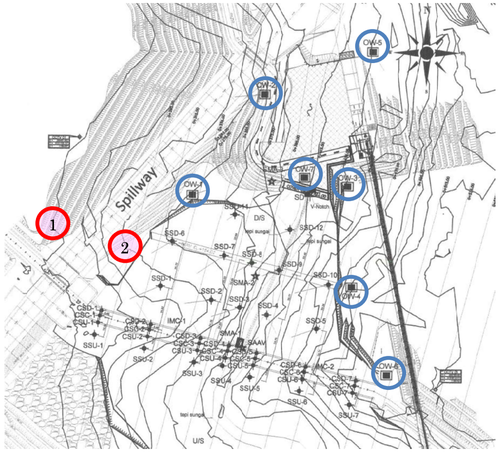
(3) to compile and reevaluate data such as the test results of the filling material (especially the permeability) and the settlement of the embankment body including asphalt core and filter zone.

3.2 Boring Survey of Asphalt Core

The leakage from the asphalt core has not been confirmed and discussed, it is recommended to carry out a boring survey of the asphalt core.

3.3 Additional Groundwater Observation Well

Seven (7) ground water observation wells have been installed at site but there is only one well on the left abutment. Since the water level is lowered at present, it is recommended to install several observation wells on the left abutment (refer to figure below).



3.4 Additional Data Analysis

The evaluation described in this note is conducted based on the results of the technical discussions with KKB/BTB and the materials distributed only. It is necessary, therefore, to finalize this evaluation note after compiling and reviewing the following detailed information.

- ✧ In-situ Test Results (Actual Embankment Permeability of Each Zone)
- ✧ Monitoring records of Multilayer Settlement Gage
- ✧ Detail Seepage Measuring Data (in particular before construction)
- ✧ Core photos and logs of along the dam axis and rims of the dam.
- ✧ Relation between reservoir water level and each ground Water Level.
- ✧ Manshette grouting construction data

These materials should already be organized at the site office, so please provide them.

Reference Data

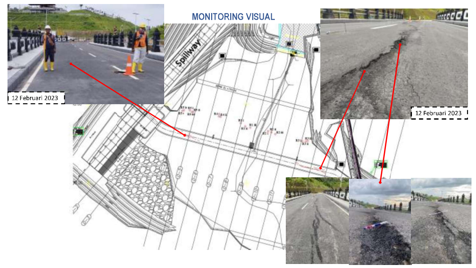


Figure 1 Crack on the Dam Crest Road

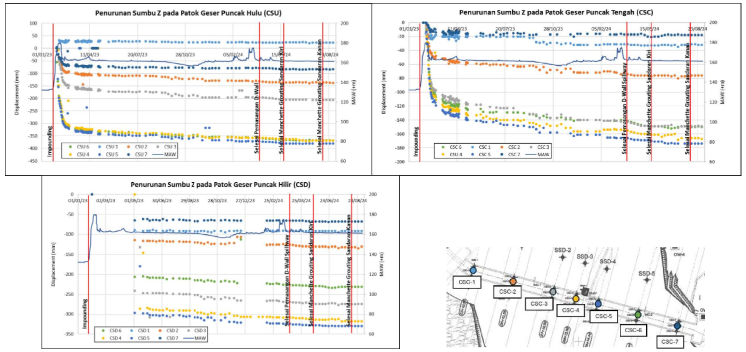


Figure 2 Settlement at the Dam Crest

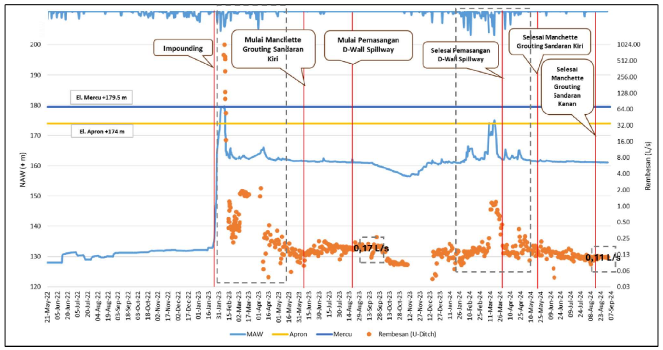


Figure 3 Monitoring of Seepage Water

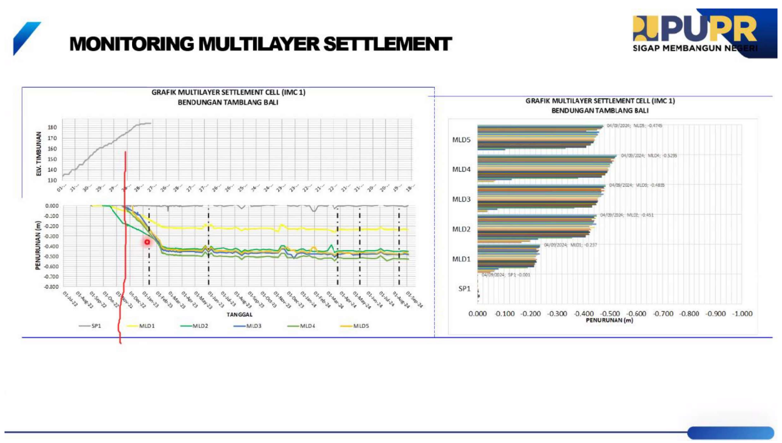


Figure 4Maltilayer Settlement Gauge

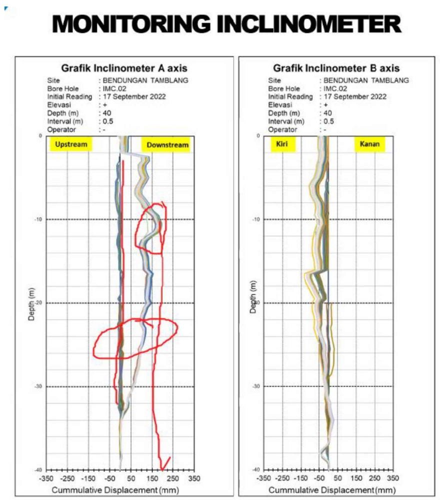


Figure 5 Inclinometer

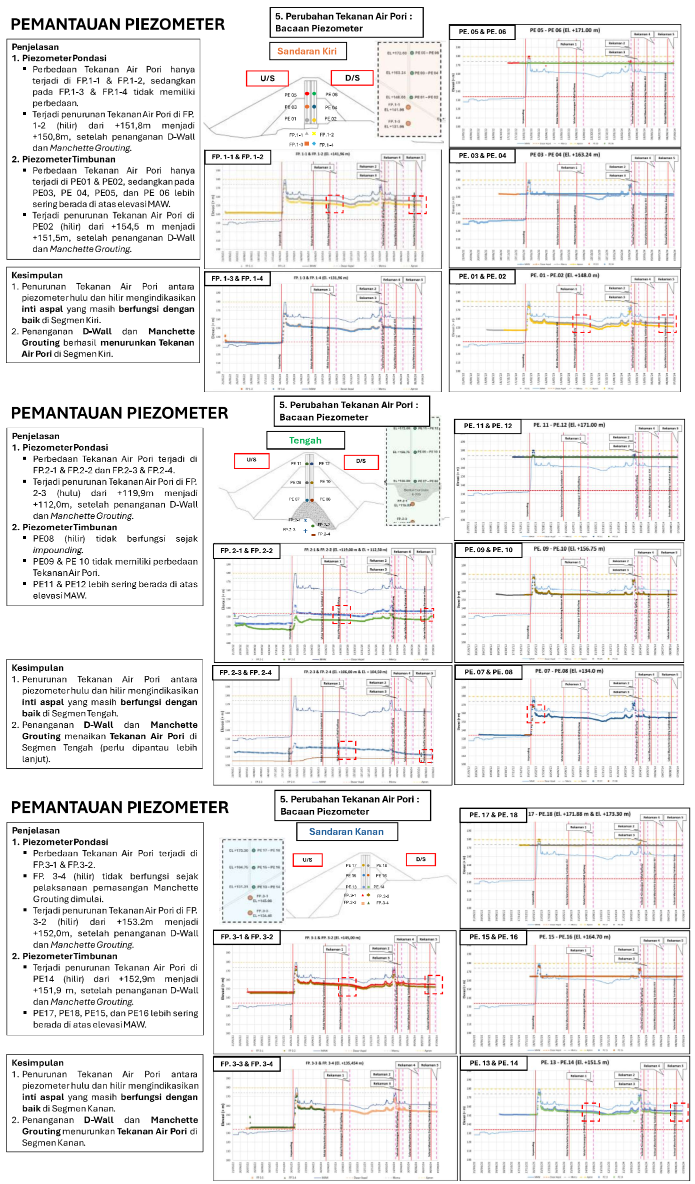


Figure 6 Piezometer

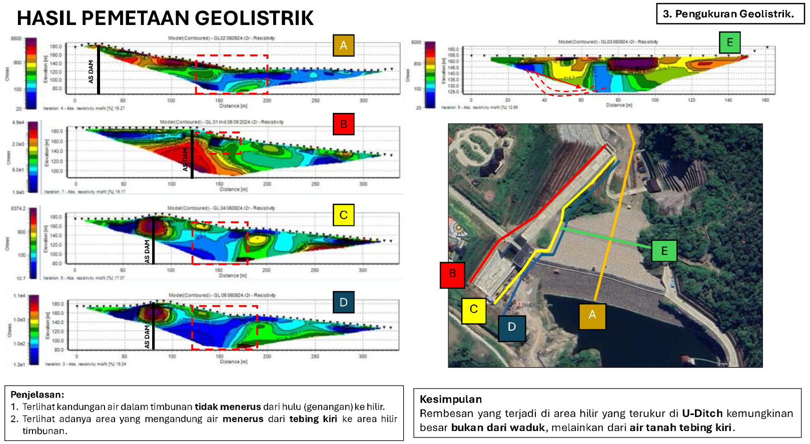


Figure 7 Geo-Electric Survey

Discussion on Meninting Dam

JICA Dam Construction Advisors
Hiroshi Shimizu

1. Meninting Dam General Information

1.1 Location:

Figure 1.1 Jenelata Dam Location Map

1.2 TECHNICAL DATA

Technical data of the Jenelata Dam is as follows:

Reservoirs

Body Dam

2. Geology

2.1 Reservoir Area

2.2 Damsite

3. Comments

The Para-Plenary Meeting for Impounding of Meninting Dam was conducted on 30 July 2024.

At the time of the inspection, the construction progress had reached 3.833%. Ongoing work includes excavation of the main dam foundation, access roads for facility buildings, etc. Based on the results of the inspection and discussions in the field, the following are recommended as follows:

The main discussion in the meeting were as follows, and I proposed responses to them.

Comment/Answer/Suggestion in the Meeting

	Comments by DSC	Answer by the Consultant	Advise by Dam Advisor
Q1	What are you going to conduct a slaking protection measure on excavated surface of tuff and claystone that are prone to slaking (they start slaking in a few days)?	Please advise us appropriate countermeasures.	⇒ leaving about 50 cm of cover rock and protecting the excavation surface with shotcrete, and so on. These measures had been conducted at BiliBili dam construction.
Q2	The prince foundation includes tuff breccia and the boundary between tuff and mudstone strata with different strengths and remaining colluvial at some parts. Have you analyzed the deformation?	In China, it is generally applicable this kind of foundation as an prince foundation.	⇒ Check the deformation module and future deformation amount with a loading test.
Q3	How did you decide on the tuff breccia design value for the gravity dam foundation? Have you conducted a block shear test?	The design parameter has determined applying laboratory tests results. Insitu direct block shearing test is not conducted.	⇒ Insitu direct shearing test can be performed without an audit, so it shall be conducted and re-confirm the validity of applied design parameters.
Q4	The dam is zoned applying a rock zone (basalt) for the upstream part of the main body and random zone (tuff breccia) for the downstream. Tuff breccia, however, is prone to fine graining after compaction. Please check the crushing condition, permeability, etc. with a trial embankment?	The Contractor will carry it out before commencement of embankment.	⇒ Conduct a trial embankment as early as possible. Use the construction equipment used on-site.
Q5	Single row curtain grout is applied in the design even in the soft rock area. What is the basis for a single row?	In China, single raw is generally applied considering economic advantage.	⇒ The number of curtain grout rows is determined based on the rock characteristics such as maximum injection pressure or critical pressure of rock, amount of cement milk that can be injected, and distribution of cracks in the foundation rock. In particular, in the case of soft rock, cement milk can not penetrate easily, so it is taken a method to ensure the water-stopping effect by a thicker curtain width. From a safety perspective, it may be better to install two rows. Please check with a grout test.

Discussion on Sidan Dam

JICA Dam Construction Advisors
Hiroshi Shimizu

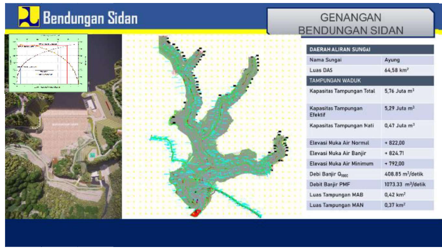
1. Sidan Dam General Information

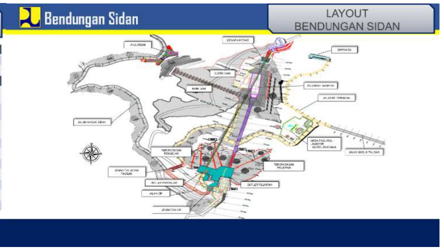
1.1 Location:

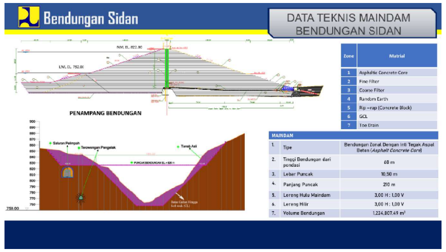
Figure 1.1 Sidan Dam Location Map

1.2 TECHNICAL DATA

Technical data of the Sidan Dam is as follows:







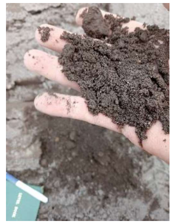
2. Comments

The Pra-Plenary Meeting for Impounding of Sidan Dam was conducted on 26 September 2024.

The main discussion in the meeting and my recommendations are as follows.

2.1 Random Material

(1) Gradation of Random Material

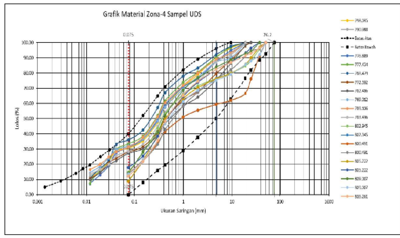


The random material contains about 30% of silt/clay (0.074mm under), and 50% of sand (0.074-2mm). Generally, materials used as core materials contain more than 15% of silt/clay, so this random material used at Sidan Dam is considered to be almost equivalent to the core material. The coefficient of permeability is $k=10^{-7}$cm/sec in laboratory tests, $k=10^{-5}$cm/sec in large-scale tests, and $k=10^{-4}$cm/sec in the embankment test site.

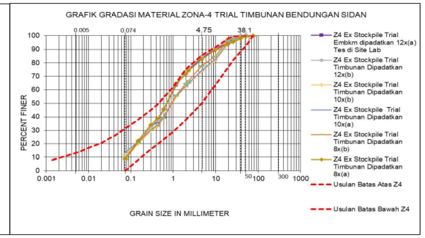
The random material confirmed at the site in July was mostly sandy soil, as shown in the photo on the right, but the test results during construction showed that it contained a considerable amount of silt/clay (about 30%).

Inferring the properties of this material, the following issues were anticipated.

- For materials whose consistency is judged as non-plastic, it is acceptable to include cohesion in a design parameter or not?
- It is anticipated that the compaction of random materials was carried out appropriately. The grain size of the material for which the compaction standard was determined, is 10% of silt/clay and 50% sand, while the acutual embankment material contained 30% of silt/clay with a significant difference in the fine particle content.
- If the material has a characteristic of sand, there is concern about settlement when impounding and the sweeping away of the embankment materials when the water drawing down.



Actual Zone 4 Materials in Construction



Material used for Trial Embankment

(a) Cohesion of Random Materials

Even for granulated materials without cohesion, when a triaxial compression test is conducted and the strength is evaluated as a straight line

($\tau = \tau_0 + \sigma \tan \varphi$) using the Mohr circle, the initial strength of τ_0 can be applied even if there is no restraining force, due to the interlocking of the materials. This phenomenon is recognized for rock materials, and when showing its strength, it is expressing by polyline (2 or 3 lines) or an envelope curve of the Mohr circle ($\tau = a\,\sigma^b$) in some cases. In the case of sand, this interlocking force is small, so τ_0 hardly appears in the test results. On the other hand, if there is a sufficient amount of silt/clay containing, the cohesion of the silt/clay can be expected.

Therefore, it is recommended to carry out a triaxial compression test and evaluate the results, then decide whether to use τ_0 in the strength or not, or to apply polyline/envelope curve.

(b) Compaction

The random material used in the Sidan Dam construction has a difference in the silt/clay contents between the material for which the compaction standard was determined in the Trial Embankment and the material actually used for the embankment. Therefore, it is essential to re-compile the test results during construction, such as the density, unit weight, and hydraulic conductivity, and to confirm that appropriate compaction has been performed before impounding.

(c) Sweep Out of Embankment Material

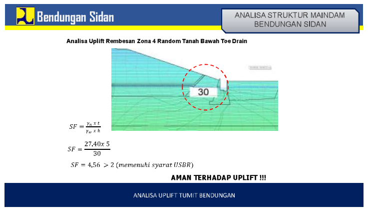
Regarding the upstream slope, a filter was placed under the concrete slab which is substitute for riprap, to prevent the sweeping out of the embankment material (random material). If the filter grain size is appropriate, material sweeping out may be prevented when the water level drops. The filter grain size shall be checked with the filter criteria.

On the downstream slope, it is thought that it will be eroded relatively easily if the slope is not protected, considering the grain size of the random material. It is recommended that measures that can be taken now, such as sodding or installing drainage ditches, be taken.

(2) Uplift at Toe of Dam

The results of the seepage analysis using Geo-Studio show that a large uplift of 30 ton/m2 acts at the downstream toe. In response to this result, some issues have suggested such as the initial conditions of the seepage analysis may be incorrected (the analysis range is narrow, the permeability coefficient ($k=10^{-3}$cm/s) is too large, etc.).

Simple consideration in terms of a flow-net, a flow line of water surface at the upstream end should be reached to a drain in the random zone or toe of embankment and the equipotential line will be 0.



It is recommended to review the analysis conditions and recalculate.

2.2 Others

(1) Control of Impounding Speed

Due to concerns about risks with asphalt concrete cores and sandy random materials in Sidan Dam, it is recommended that water impoundment proceed as slowly as possible. The current plan is to impound at an average speed of 1 m/day. It is recommended that the outlet facilities (Hydromechanical Works) be installed with time to spare, and even after discharge becomes possible, the reservoir should be impounded slowly while closely monitoring the behavior of the embankment.

The rainy season will soon start, so if reservoir impounding begins at this time, it is supposed that it will be difficult to control the water level in an emergency. Please take this risk into consideration before impounding.

(2) Monitoring of Dam Behavior

Although there are no thermometers or strain gauges in the asphalt core, the strain stress in the asphalt core shall be estimated based on the record of settlement gauge and inclinometer and confirm the safety of asphalt core.

Furthermore, after impounding, be inspected the dam carefully/continuously and confirm that there is no signs of cracks that could lead to leakage from the core or core base interface.