Republic of Indonesia, Ministry of Public Works
Directorate General of Water Resources

Indonesia
Expert Dispatch Scheme
Dam Design and Construction
Advisory Services

Completion Report
Data Book

February 2025

Japan International Cooperation Agency

CTI Engineering International Co., Ltd.
CTI Engineering Co., Ltd.

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Data Book

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2. ***Minutes of Meetings***
3. ***Presentation Materials in Seminar***
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Site Investigation Report

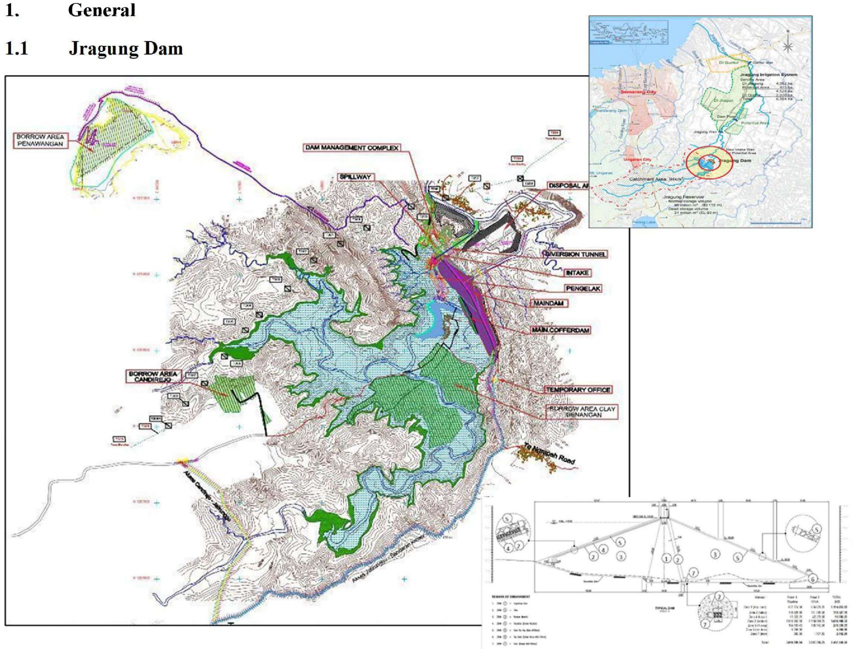
Hiroshi SHIMIZU
JICA Dam Construction Advisor

Name of Dam : Jragung Dam

Date : June 14, 2023

1. General

1.1 Jragung Dam



(1) Hydrological Data

- River Name : Jragung
- Catchment Area : 94 km²
- River Length : 35 km
- Average Annual Rainfall : 2,479 mm
- Average Annual Discharge : 114 million m ³
- Q_{25} Flood Discharge (*inflow*) : 280 m³/sec
- Q_{1000} Flood Discharge (*inflow*) : 825 m³/sec
- Q_{PMF} Flood Discharge (*inflow*) : 1,580 m³/sec

(2) Reservoir

- Total Reservoir Capacity : 90 million m³
- Effective Reservoir Capacity : 69 million m³

- Dead Reservoir Capacity : 21 million m³
- Low Water Level (LWL) : +93.00 m. asl
- El. normal water level (NWL) : +115.00 m. asl
- El. 1000 year flood water level : +117.28 m.asl
- El. PMF flood water level : +118.65 m.asl

(3) Diversion Facilities

- Design Discharge (Q_{25}) : 181.74 m³/sec
- Elv. Temporary cofferdam peak : +76m.dpl
- Temporary cofferdam
  - height : 11 m
  - length : 501 m
  - crest width : 6m
  - upstream slope : 1: 2,0
  - downstream slope : 1: 2,0

(4) Main Dam

- Type : Zoned Type Random Fill-dam with Core
- elv . peak dam : +119,5 m. asl
- elv . base foundation : +60,0 m. asl
- Dam height : 59.5 m (from the bottom of the foundation)
- Crest Width : 10 m
- Dam Length : 1305.3 m
- Upstream slope : 1:3,0
- Downstream slope : 1:2,5

(5) Spillway

- Type : Abundant free
- Type threshold : Ogee type II
- elevation threshold : +115 m
- Total guard height : 4.50 m
- Overflow width : 60 m, shaped arch
- Lighthouse length : 12.30 m
- The total length of the overflow : 439.00 m
- Flood discharge - Inflow
  - ✓ Q_{100} : 378.00 m 3 / sec
  - ✓ Q_{1000} : 822.00 m ³ / sec
  - ✓ Q_{PMF} : 1,580.00 m ³ / sec
- Flood discharge - Outflow
  - ✓ Q_{100} : 170.00 m 3 / sec
  - ✓ Q_{1000} : 446.00 m 3 / sec
  - ✓ Q_{PMF} : 981.00 m 3 / sec

1.2 Progress of Works

Based on the information obtained during field inspections, the physical progress of the construction of the Jragung Dam has reached 32.51% and the financial progress is 23.40%. Ongoing works are main dam embankment, intake, Spillway, dam operation office, hydromechanical work and inspection road work.

2. Issues Discussed

2.1 Deformation of Inlet and Outlet of Diversion Tunnel

Deformation occurred during this rainy season at the Diversion Tunnel exit.

The Site Engineers proposed thickening the lining concrete and narrowing the tunnel diameter. But in this case it is necessary to reevaluate the flow capacity of the tunnel.

The most important issue, however, is whether the tunnel can maintain its structurally and sufficiently stabilized. Generally, during tunnel excavation, the "Rock Mass Rating" is reviewed at any time, and the number and length of rock bolts and the interval of Steel Rib Support are changed as necessary. In this case, it is necessary to confirm whether the RMR's review was done appropriately. Since the deformation is limited to the section 13m from the tunnel outlet, it is necessary to reconfirm the geological structure of this deformation area (zone).

The geological evaluation and NATM measurement results at the time of completion of tunnel excavation was requested to the Site Engineer but have not received them yet.

For repair of damaged tunnel, several construction methods such as pipe-roof can be proposed.

2.2 Design Change of Intake Structure

The intake structure was originally designed as a sloping intake type that would be installed on the foundation rock above the diversion tunnel considering the operation and maintenance of outlet facilities. However, during the foundation excavation of the inclined intake structure, a slope failure occurred on the excavation surface. According to the Site Engineer, the cause of the collapse was that the slope of the excavation was too steep (H:V=1:0.7). For this reason, the design of the water intake facility was changed from the inclined intake to tower type. Three alternatives have been examined by the Site Engineer.

In the revised plan, the intake tower is placed at a position where a CM-class foundation can be secured, and the plan is to connect the intake tower and the crest of the dam with a management bridge.

Points to note in the design of intake tower type are as follows.

- ✧ Since the height of the tower will be about 40 m high, the foundation of tower shall be enough strong
- ✧ The inlet of the tunnel is deformed, it is essential to confirm the stability of the tunnel inlet and provide measures to enforce the tunnel such as filling up the tunnel with concrrete.
- ✧ Considering the operation and maintenance of Bulkhead Gate, the access to the intake tower shall be considered.
- ✧ For reference, one of the special method of operation of Bulkhead is applied to the Cirata Dam. Please check it.

2.3 Slope Failure at Intake Slope

The surface slope sliding occurred at the upper portion of the excavated slope of original inclined intake structure. It is supposed that the sliding occurred at the surface of the slope and the slip line may be not deep. However, the sliding trace is remaining at the excavation slope surface so that the sliding portion must be protected from the further sliding after impounding.

In accordance with the information form the Site Engineer, the slope with H:V=1:1.0 keep stability so that it is supposed that the slope failure occurred due to steep slope (H:V=1:0.7).

The protection measures against a further sliding shall be provided. It is considered that counterweight by means of embankment materials (random material) or slope protection/frame works with rock-bolts.

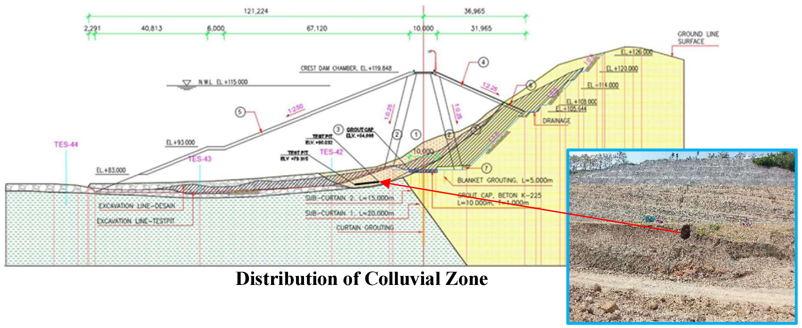
For design of protection works, followings shall be verified:

- ✧ Sliding character (it is supposed to be a surface sliding)
- ✧ Stability of the slope in future (weathering/slaking of claystone on the slope surface),
- ✧ Parameter of sliding mass (weathered clay stone),

Considering the permanent protection of slaking of claystone om the slop surface, it is supposed that the counterweight method may be preferable. The counterweight embankment may be affected the intake structure, so the area to be placed the counterweight shall be examined detail and designed retaining wall surrounding intake if necessary.

2.4 Colluvial Zone on Dam Foundation (Right Abutment)

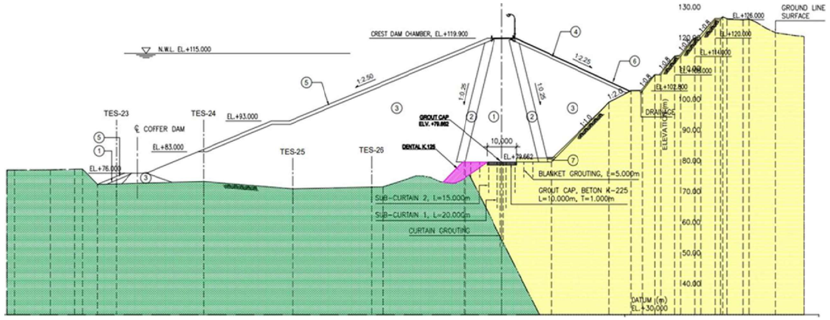
After excavation of the dam foundation on the right abatement, a colluvial zone has been observed. The Colluvial zone is judged as an unsuitable material for dam core foundation so it must be removed. In case the Colluvial zone on the foundation is removed, the shape of foundation will be inclined suddenly at around one third upstream of core foundation zone.



Distribution of Colluvial Zone

One of the countermeasures is to finish the foundation excavation surface smoothly so as not to leave sudden changes in the core foundation surface. However, since the Grout Cap Concrete has already been placed and the grouting has been completed, the best way at this time is to replace the excavated Colluvial zone with concrete and create a flat core foundation.

The Consultant has already conducted a structural analysis of the dam stability under the conditions of whit/without of the concrete replacement, and has confirmed that the replacement with concrete will ensure the stability of the dam.



Proposed Dental Work (Replacement with Concrete)

In general, immediately after finishing the foundation excavation, the Site Engineers shall inspect the actual condition at site and make judgement whether the rock condition is good for dam foundation or not. In this case, before confirmation of the condition at site, the cap concrete was placed and grouting works were commenced remaining this colluvial zaon in the dam foundation.

It is essential that the engineering judgement before commencement of following works when technical issue found.

2.5 Sliding of Right Bank of Spillway Connecting Channel

At the left bank of the river connecting channel of the spillway, trace of sliding is observed and it is anticipated that excavated slope will slide. The Site Engineer proposed straitening the channel alignment to escape the colluvial area.

For implementation of this modification, the hydraulic condition at the connection point with old river must be evaluated and some measures such as bank protection at water colliding portion or curvature of channel shall be provide, if necessary.

Site Investigation Report

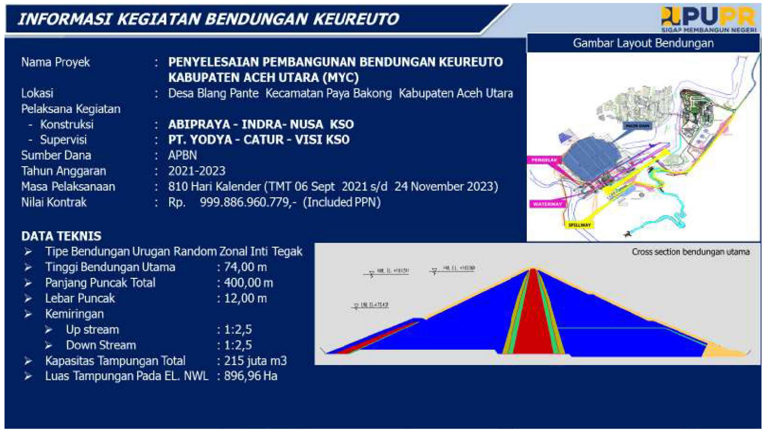
Hiroshi SHIMIZU
JICA Dam Construction Advisor

Name of Dam : Keureuto Dam
Date : June 20, 22, 2023

Checklist of Issues

1. General

1.1 Keureuto Dam



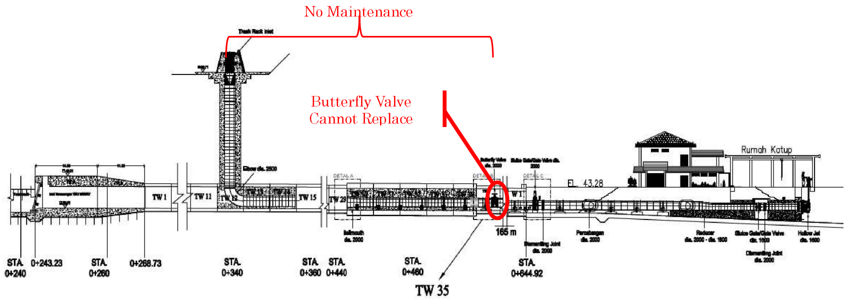
1.2 Progress as of June

2. Issues Discussed

2.1 Design Change of Intake Facilities

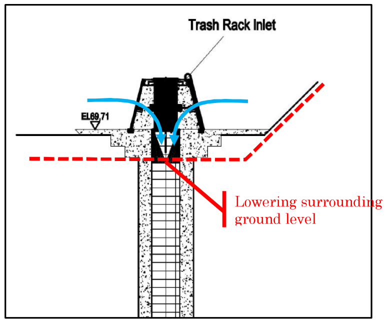
(1) Type of Intake Facilities

In the original design, the intake system was designed as inclined type, however, the type was changed to drop type due to the bad condition of the foundation. The structure was already constructed as shown in the photo. Due to the change of structural type of intake, bulkhead gate cannot be installed at the inlet portion, so a butterfly gate is currently proposed for maintenance of outlet facilities by the project office. The reason why the bulkhead gate cannot be installed is a long operation bridge required. The comment from DSC is the maintenance at inlet will not be considered. Maintenance from the entrance to the Butterfly Gate cannot be made permanently.



Since the maintenance of inlet and pipe/tunnel from inlet to butterfly valve and rehabilitation/replacement of butterfly valve cannot made, it is necessary to provide any measures minimizing the risk of damages in this portion.

To minimize damages in this portion, it is important to protect inflow of solid matters such as soils, sand and stones, floating logs and so on. At present the inflow portion is backfilled up to the same elevation of inflow level. It is recommended that backfill soil excavate to protrusion the inflow portion.



In addition, please discuss with gate supplier about gate maintenance for prolong the gate's life and replacement method of gates, in particular butterfly gate.

Please remind that the bulkhead gate at the inlet portion is very important for the maintenance/rehabilitation of the outlet facilities.

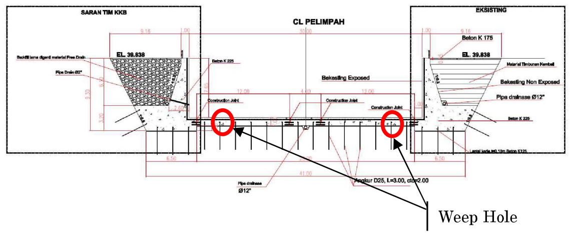
2.2 Spillway

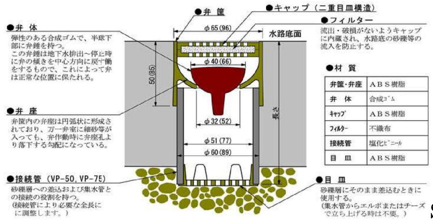
(1) Drain Pipe Under Spillway Slab

Drain pipe under the spillway slab in the upstream and downstream directions are usually arranged at a pitch of about 1 every 10 to 15 m in order to ensure reducing of uplift pressure loaded to the bottom of the slab. In the spillway of Keureuto dam, the drain pipe in upstream and downstream direction is installed only one for the 30m wide channel so the DSC commented that to provide additional release pipe.

Seepage analysis shall be conducted to confirm the uplift force acting on the slab concrete, and if necessary, weephole boring will be conducted from above the slab concrete to release groundwater. It seems that the drainpipe perpendicular to the axis of spillway is not installed, so two holes will be drilled on the upstream side about 1m from the joint perpendicular to the axis. A depth of about 2 m may be sufficient.

A weephole with a backflow prevention device with a filter is installed on the top surface of the holes to prevent the clay/sand being washed away from the foundation and counterflow from spillway channel to foundation. Many types of products of weepholes with backflow prevention are available on the market.





Sample of Weephole

(2) Leakage from Side Wall and Channel Slab

As for the cause of the water leakage, it is supposed that in addition to poor construction in concrete placement and installation of the water-stop, there were also issues with the design of the drain system at the bottom of the slab and the back of side-wall.

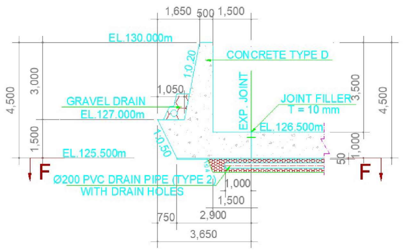
In general, drain grid system under the slab concrete and back drain behind the side wall with weep holes are designed but in Keureuto Dam, under slab drain is provided only one line at the center of 30m wide slab, and there is no drain system behind the side wall.

Since the spillway concrete was already placed, ground water may be released via weep holes newly drilled on the slab concrete and side wall concrete. Location of weepholes are;

- Slab concrete : as mentioned above section
- Side Wall : one hole par 5m2

Diameter of drain holes are 5cm.

It is necessary to pay attention on design of drain system of water-related structure.



Sample of Spillway Drain System (Jatibarang Dam)

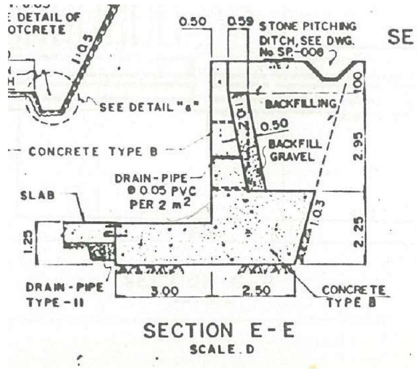
(3) Boulder/Sand/Clay flow into the Spillway Channel

Many boulders/sand/clay observed on the spillway channel and in the stilling basin. The site engineer said that boulders and sediments were come into the channel from both side slope of spillway channel. Such solids (boulders and sediment) will be a cause of erosion of spillway concrete. It is important to minimize the flowing the boulders and sediments into spillway channel.

One of protection method is lowering the backfill surface, maybe 1 m below from the top of side wall so as to the boulders and sediments can be stopped on the backfill portion. When the backfill surface is lowered, it is anticipated that rainwater runs on the surface of backfill and erodes the backfill material. As a countermeasure against an erosion of backfill material by rainwater, the spillway channel backfill surface was covered with concrete stairs to protect the erosion and dissipate the energy at the Jatibarang Dam. The stairs is utilized as maintenance path.

The other way is installing drainage channel on the surface of backfill. The water can be flow down the sediments with downstream. Bili-Bili dam applying this system since the slopes on both sides of the spillway channel are composed with solid rocks (diabase/diorite).

Jatibarang Dam Spillway



Bili-Bili Dam Spillway Side Wall

(4) Right Side Wall of Stilling Basin

It is supposed that the right wall of stilling basin was not constructed as design as shown in photo below. Sediments has flown into stilling basin from this portion.

It is recommended that this portion should be repaired as designed.

2.3 Seepage Measuring Device (V-Notch)

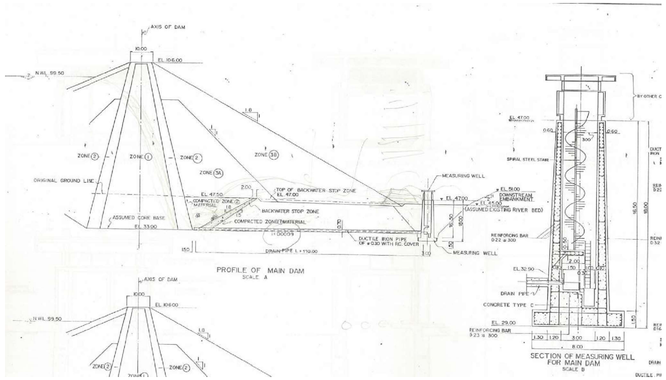
The Site Engineer proposed to raise up the position (elevation) of seepage measuring device (V-Notch) to flow the seepage water by gravity, because the downstream water level is higher than designed seepage measuring device.

However, it is not recommendable because a seepage water collecting system (backwater stop zone, refer to following figure) was not installed in Keureuto Dam.

Since dam embankment works has been conducting, it is impossible to change system. So that a proposed countermeasure is;

- ✧ Set the V-Notch at original elevation,
- ✧ Provide Measuring Well Tower to protect the river water flow into V-Notch, and
- ✧ Install a drain pump which runs automatically when water level of the drain pit (downstream of V-Notch) reach to the setting position.

The capacity of pump shall be set depending on the expected seepage discharge examined by means of seepage water analysis including rainwater on the downstream slope.



Bili-Bili Dam Seepage Measuring Device

It shall be reminded that seepage discharge when it is measured during rain, the amount of measured discharge is including rainwater.

1.3 Grouting Area and Results

In the presentation at site, the Consultant presented the Lugeon Map before and after grouting works. The maps show that the Lugeon value of all curtain grouting area before grouting were more than 20Lu. However, the depth of curtain grouting on the right bank is much shallow than left bank. It is necessary to recheck the adequacy of setting of curtain grouting area.

Furthermore, check holes, crossing one grouting pattern (12 m?) and deeper than 1 stage (5 m) from the curtain grouting area, might be conducted after completion of grouting but there is no data below curtain grouting area as of now.

It is required to prepare an evaluation report of grouting containing a cement take, injection time and etc. as well as Lugeon value.

Site Investigation Report

Hiroshi SHIMIZU
JICA Dam Construction Advisor

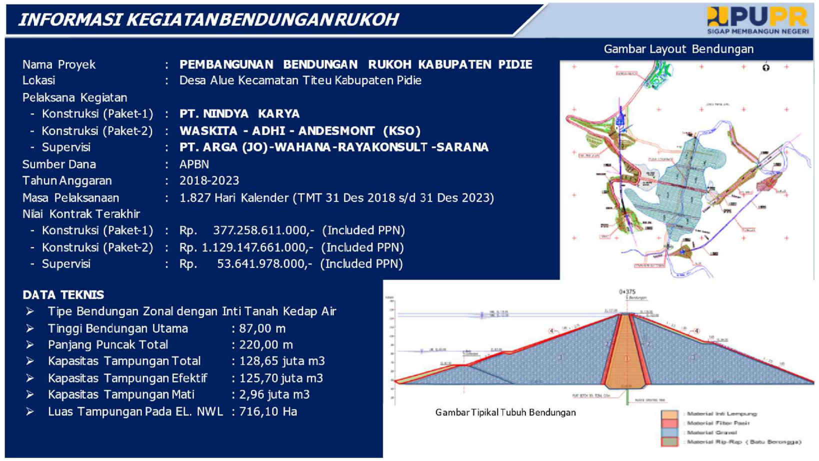
Name of Dam : Rukoh Dam

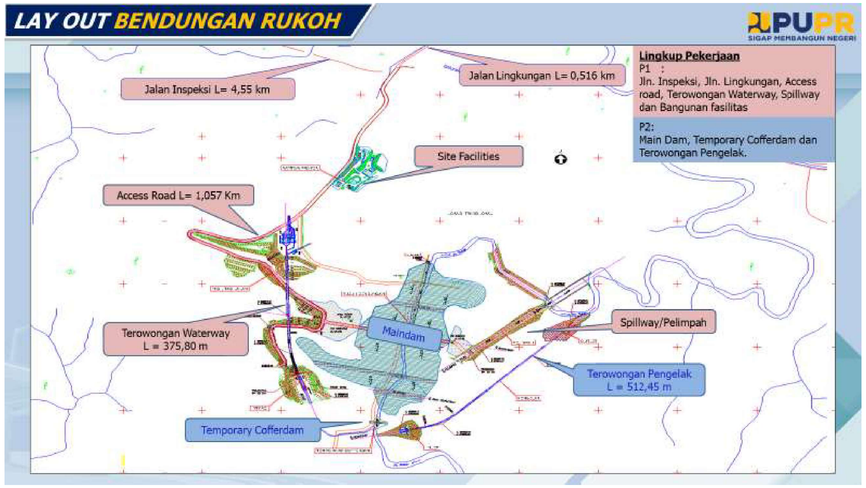
Date : June 21, 2023

Checklist of Issues

1. General

1.1 Rukoh Dam





1.2 Progress/Schedule as of June 2023

No data

Dam embankment works will be started from July and completed at November 2023.

Initial impounding will start within this year.

2. Issues Discussed

2.1 Spillway

(1) Under Slab Drain

Under slab drain in upstream and downstream direction is not installed while the cross drains are installed. The water in the cross drain will not be able to flow down and uplift pressure under the slab remains.

To settle this problem, the Site Engineer started to install drainpipe outside of the side wall connecting cross drain (refer to photo in right). The details of the works are unknown.

To install drainpipe, the foundation of side wall is excavated, therefore it is anticipated that the foundation will be damaged. The works should be carried out very carefully and back fill must be conducted adequately.

The easiest way to release uplift pressure is drilling weepholes on the cross drain and installing a backflow prevention device at each weephole.

2.2 Seepage Measuring Device (V-Notch)

Issue of seepage measuring device is same as Keureuto dam.

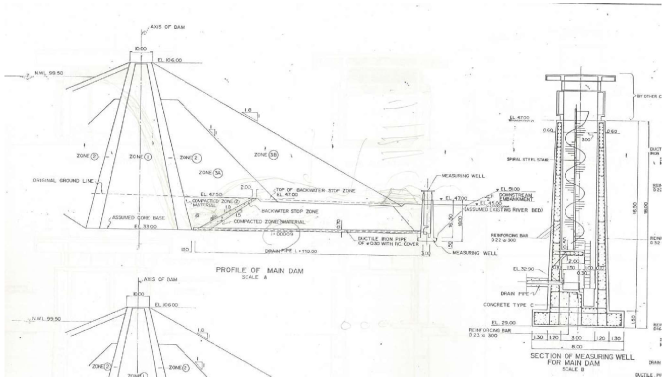
The Site Engineer proposed to raise up the position (elevation) of seepage measuring device (V-Notch) to flow the seepage water by gravity, because the downstream water level is higher than designed seepage measuring device.

However, it is not recommendable because a seepage water collecting system (backwater stop zone, refer to following figure) was not installed in Lukoh Dam.

Following current design, the seepage measuring device will be designed as follows;

- ✧ Set the V-Notch at original elevation,
- ✧ Provide Measuring Well Tower to protect the river water flow into V-Notch, and
- ✧ Install a drain pump which runs automatically when water level of the drain pit (downstream of V-Notch) reach to the setting position.

The capacity of pump shall be set depending on the expected seepage discharge examined by means of seepage water analysis including rainwater on the downstream slope.



Bili-Bili Dam Seepage Measuring Device

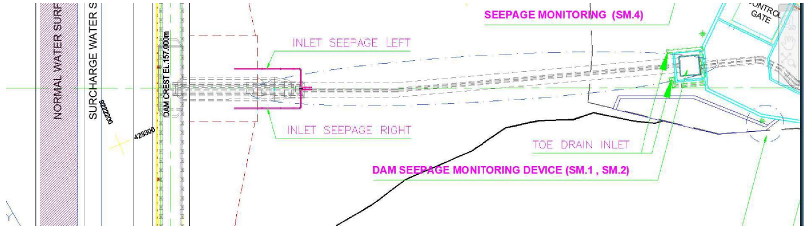
It shall be reminded that seepage discharge when it is measured during rain, the amount of measured discharge is including rainwater.

【Alternative】

Since the dam embankment is not started, it is possible to change design of system as below:

- ✧ construct seepage water collecting system (backwater stop zone)
- ✧ set the pipe higher position of downstream water level.
- ✧ at the downstream of V-notch, provide pit and at the outlet of the pit, install flap gate to protect the backflow from the river.

This system was applied to the Jatibarang dam.



2.3 Embankment

(1) Embankment Schedule

According to the Site Engineer, the embankment works will start from July 2023 and completed at the end of November 2023 so that the working day will be 150 days only. Since the dam height is 87m, daily embankment height shall be 60 cm/day. The core embankment will be performed as below:

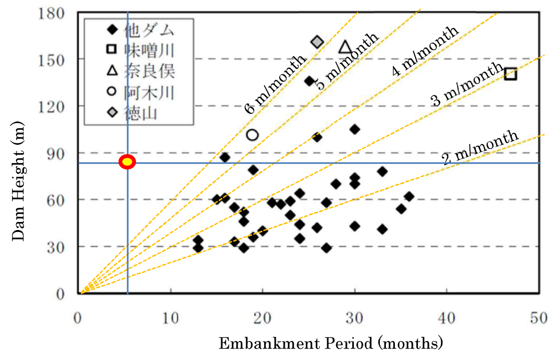
- ➢ Spreading core material with a thickness of 30 cm,
- ➢ Thickness after compaction is 25 cm.

Therefore, even if two layers of core were embanked a day, this plan will not be able to achieve this plan. Furthermore, extra embankment will be required additionally.

This plan, therefore, is unrealistic because Ache region will enter the rainy season from now on and the workable days for core embankment will be limited in order to ensure the quality of the core (the number of workable days that is good for core embankment will be shortened).

The implementation plan should be revised after considering the workable days fro core embankment, settlement of core due to rapid construction of core and the time required for field tests, especially field permeability tests.

Following figure shows actual records of embankment speed in Japanese fill type dams. As shown in the figure, the fastest embankment speed in Japan is 6m/month, but at Keureuto Dam, it is planned to be 17 m/month.



Relationship between Dam Height and Embankment Period (actual record in Japan)

(2) Embankment Material (Random Zone)

In the current criteria for liquefaction judgment, the grain size range of the soil to be liquefied is defined as those with a fine grain content (Fc) of 75 microns or less, which is a silt and clay, of 35% or less. As far as I could see at the site, fine particles contents are very small and mainly composed of coarse sand and boulders, and there was concern about the risk of liquefaction of the embankment. The Site Engineer should confirm the possibility of liquefaction in the dam body prior to the commencement of random fill works and take countermeasures, if necessary.

In Japan, as a countermeasure against liquefaction of the dam body, the relative density (Dr) that does not cause liquefaction is set using the results of the liquefaction characteristics test, and it is managed by the compaction criteria. The other measure is providing drain in the random zone to ensure the drain in the dam body. There is also an example in which the dam type is CFRD to prevent saturation in the dam body.

Activity Report

Technical Discussion on Damsite Selection at Sintang District, Kapuas River

Hiroshi SHIMIZU
Naoya MIZUNO
JICA Dam Construction Advisor

Name of Dam : Sintang Dam, Kapuas River

Date : July 18, 2023

1. General

On July 18, 2023, technical discussion on Damsite Selection along the Kapuas River at Sintang Regency, West Kalimantan in the following manner.

Place : Jatigede Dam Meeting Room, Floor 7, SDA patimura, PUPR

Time : 09:00 – 12:00 (WIB)

Topics : Discussion on Alternative Damsite Selection of Sintang Bedungan Sintang District, West Kalimantan Province

2. Conclusion of Discussion

a. On the left abutment of Alternative Damsite 4 (ALT4), a fairly/thick soil deposit or hardly weathered rock is observed.

b. Geologically, Alternative Damsite 5 (ALT5) is relatively better than ALT4, since metamorphic rocks which easily become weak zones (weathered) are distributed ay ALT4 and this kind of rocks are not found at ALT5.

c. As mentioned above, it is supposed that ALT5 is somewhat better than ALT4 from the geological viewpoints. The further detail geological survey will be conducted at ALT5.

d. Need a more detailed geological information at damsite (strengthening argument) to judge an applicability of Dry Dam or Multipurpose Dam

e. Generally, the topsoil will not be too deep, check again its distribution and condition of the topsoil (colluvial or weathered).

f. Check the ownership, available volume and access of the proposed rock quarry site of embankment materials. Besides, try to find an alternative quarry sites inside of the proposed reservoir area.

g. The proposed core material has a fairly high water content compared to its OMC value. Need more explanation about the material, whether it will be mixed with sand or with other handling.

h. Since the riverbed material is coarse sand, there is a potential for erosion on the tunnel invert. So it is necessary to be taken into consideration this issue in the tunnel design.

i. When the required quality of dam foundation as CL class rock, the excavation becomes too large/deep. Reconsideration to change required quality of dam foundation depending on the height of dam. It is supposed that the dam foundation may be placed on soil with an SPT value of > 30 or SPT > 50 to reduce excavation.

j. Please pay attention to the environment because it has an impact on endemic animals in Kalimantan in particular.

k. It is supposed that the effectiveness of flood reduction by dam is relatively small, the consultant needs to explain on this issue to the public to obtain a consensus with local people.

l. The consultant applied the OM cost as 0.3% of construction cost in the economic evaluation, but usually 1% is applied in the PUPR projects. Please show the evidence of the 0.3%.

3. Evaluations & Comments by Dam Advisors

3.1 Planning and Design Issues

On November 2021, immediately after flood at Sintang City, Mr. Shimizu visited at Sintan city together with the Director of BINTEK and Head of Dam Technical Center and reviewed the existing plan. The conclusion of review of the ongoing plan are:

- ✧ Hydrological and Hydraulic Analysis (rainfall and runoff) to confirm the character of flood in the Kapuas River basin including inundation analysis;
- ✧ Confirmation of flood control effects by flood storage facilities such as dam and retention ponds;
- ✧ Confirmation of material availability, in particular the quarry site of sandstone and the borrow pit of clay for core materials;
- ✧ Numerical hydraulic analysis to confirm the effects of the Sanggau shortcut channel and the Melawi River Diversion Channel; and
- ✧ Environmental Impact Assessment (especially for rare and endangered species and river water pollution).

The Investigation Report at Sintang, West Kalimantan is attached herewith.

3.2 Geological Issues

(1) General

Followings are comments on the ALT5 site geology:

- ➢ Topsoil generally refers to soil with a high concentration of organic matter and microorganisms in the top layer. In the core log, however, the thick soil layer including colluvial soil and weathered residual soil has been indicated as "Topsoil". It is necessary to use the term appropriately.
- ➢ It is not appropriate to indicate a soil layer as D Grade rock in the core log because soil shall not be classified by rock mass classification.
- ➢ Considering the dam height at the abutments, there is a possibility that the dam foundation of both abutments can be made a little shallower. However, it is necessary to check the permeability and durability of the foundation rock. Since the criteria of the excavation line greatly affects the embankment volume, it is necessary to consider the site condition well as well as dam height.

(2) Estimation of site geology based on core photos

Based on the condition and color of the drilled core samples, the soil layer and weathered structure were estimated. Then, the geological condition of each borehole was estimated comprehensively considering the topography and N-value.

(a) BHM5-1

(i) Structure of soil layers and weathering

- The depth of 5.3 m below the surface is strongly weathered and turns brown. In addition, the groundwater level has been confirmed at a depth of around 5.3m.
- At depths of 5.3 to 20m, weathering is observed overall, although it is not as strong as the upper layers.
- The depths of 20 to 27m show fresh colors here and there. It is estimated that weathering progressed along the fissures.
- It can be judged that the rocks deeper than 27m are fresh rocks. However, since weathered fissures are distributed up to a depth of 33 m, there is a possibility that it has high permeability.
- The section where cores are collected in the form of gravel here and there is interpreted as the part where weathering progressed along the fissures. In addition, if slickenside is observed in rock fragments in or around sections where cores are collected in the form of gravel, it is estimated that small-scale faults are distributed in those sections.

(ii) N-value

- Standard penetration test was conducted from the depth of 1.55m to 17.5m at intervals of 2m.
- The N value is 24 near the depth of 1.5m, and the N value is 34 near the depth of 3.5m.
- N-values at the depth of 5.55 to 6.0m cannot be read in the core photos.
- The N value is 45 near the depth of 7.5m, and the N value is 50 or more at the depth of 9.5m or deeper.

(iii) Synthetic interpretation

- Since a series of weathered structures are recognized from near the ground surface to a depth of 27m, almost no cover layer is distributed near the ground surface, and it is estimated to be weathered residual soil and weathered rocks. If we observe the core at the depth of 5 m or more, we may be able to recognize the structure of the rock.
- When the cover layer (alluvium) is distributed, the N value near the surface is usually 10 or less. However, high N values from near the surface support the above estimation.
- Based on the N-value, the weathered residual soil deeper than 10m depth can be used as the foundation of the pervious zone. Also, if the weathered residual soil and underlying bedrock are less pervious, they may be used as a foundation for impervious zones.

(b) BHM5-2

(i) Structure of soil layers and weathering

- The depth of 27.1m from the surface is sediment rather than colluvial soil.

- At the depth of 3 m from the ground surface, the sediments are strongly weathered and brown in color. The groundwater level is estimated to be distributed around 3m depth because the core samples deeper than 3m are fresh color.
- The depth of 3 to 27.1 m is estimated to be hydrological deposits. Of these, the depth of 3 to 16.5m is mainly composed of silt or fine sand. They are estimated to have been deposited in a calm environment without currents.
- The depth of 16.5 to 27.1m is riverbed sediment containing fresh cobbles.
- Deeper than 27.1 m, the bedrock is fresh and has many high-angle fissures.
- The bedrock at the depth of 35 to 36m may have a weathered color and may have high permeability. This weathered zone may have formed before the deposition of riverbed sediments.

(ii) N-value

- Standard penetration test was conducted from the depth of 1.55m to 17.5m at intervals of 2m.
- The N value is 20 or less at the depth of 1.55 to 16m, and a tendency to increase with depth is recognized. However, it is difficult to read the exact N value in the core photos.
- The N value around 17.5m depth is 50 or more.

(iii) Synthetic interpretation

- The area around the borehole point is estimated to be an old river channel. The depth of 16.5m to 27.1m is interpreted as riverbed sediments or debris flow deposits deposited in the channel.
- After that, the area around the borehole site became a small lake, and silt to fine-grained sand with a depth of 3 to 16.5m was deposited. And now, colluvial soil from the surface to depth 3 is deposited over the lake sediments.
- Based on the N-value, the riverbed sediment deeper than 16.5m can be used as the foundation of the previous zone.

(c) BHM5-3

(i) Structure of soil layers and weathering

- From the ground surface to the depth of 7.4 m, there is brown sediment. Since it does not contain gravel, it is estimated to be weathered residual soil rather than colluvial soil.
- The rock mass at the depth of 7.4m to 17.4m have weathered fissures and may have high permeability. Since fissures deeper than 17.4m are fresh, it is estimated that the permeability of bedrock is low.

(ii) N-value

- Standard penetration test was conducted from the depth of 1.55m to 6.5m at intervals of 2m.
- The N value is 50 or more at the depth of 1.55m.

(iii) Synthetic interpretation

- It is estimated to be weathered residual soil from near the ground surface to a depth of 7.4m. Based on the N-value, it can be used as the foundation for the pervious zone if the soil near the surface is removed.
- The rock mass between from the depth of 7.4m to 17.4m is weathered and may have high permeability.
- It is necessary to set the foundation excavation line of the impervious zone considering the permeability of the bedrock at the depth of 7.4 to 17.4m.

(d) BHM5-4

(i) Structure of soil layers and weathering

- The depth of 27.2m from the surface is brown sediment and does not contain fresh gravel.
- The bedrock deeper than 27.2m has a fresh color. However, the rock mass up to the depth of 32m has weathered fissures and may have high permeability.

(ii) N-value

- Standard penetration test was conducted from the depth of 1.55m to 21.5m at intervals of 2m.
- The N value is less than 20 from the ground surface to around 7.5m depth.
- The N value around 9.5 to 11.5m depth is 20 or more.
- The N value is 50 or more at deeper than 13.5m.

(iii) Synthetic interpretation

- It is estimated that the 13.5m depth from the surface is colluvial soil, and the depth of 13.5 to 27.2m is weathered residual soil. However, around 9.5 to 11.5m in depth, the N-value is about twice that of the upper part, so there is a possibility that the sediment is different from the upper part, such as a terrace deposit.
- Based on the N-value, the above weathered residual soil can be used as the foundation of the pervious zone.
- As mentioned above, rock masses up to 32m deep can have high permeability. When examining using the rock mass as the foundation of impervious zones, it is necessary to consider the permeability and grouting efficiency of the rock.

(e) BHM5-5

(i) Structure of soil layers and weathering

- From the surface to the depth of 19.2 m, the soil is brownish and does not contain fresh gravel.
- The bedrock deeper than 19.2m has a fresh color. However, the rock mass up to a depth of 33m has weathered fissures and may have high permeability.

(ii) N-value

- Standard penetration test is conducted from the depth of 1.55m to 18.5m at intervals of 2m.
- The N value is 20 near the depth of 1.5m.
- The N value is 30 near the depth of 3.5m.
- The N value is 50 or more at the depth of 5.5m and deeper.

(iii) Synthetic interpretation

- The depth of 19.2m from the ground surface is estimated to be weathered residual soil. However, there is a possibility of colluvium up to around 5.5m where the N value is low.
- Based on the N-value, the weathered residual soil deeper than 5.5m can be used as the foundation of the pervious zone.
- As mentioned above, rock masses up to 33m deep can have high permeability. When examining using the rock mass as the foundation of impervious zones, it is necessary to consider the permeability and grouting efficiency of the rock.

A summary of the above is shown in Figure 1.

Site Investigation Report

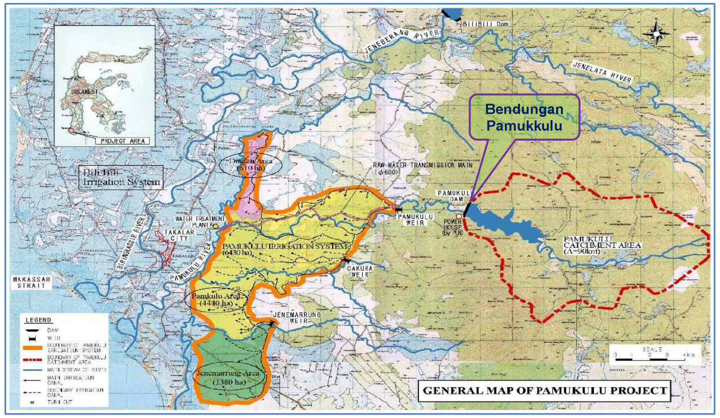
Hiroshi SHIMIZU
JICA Dam Construction Advisor

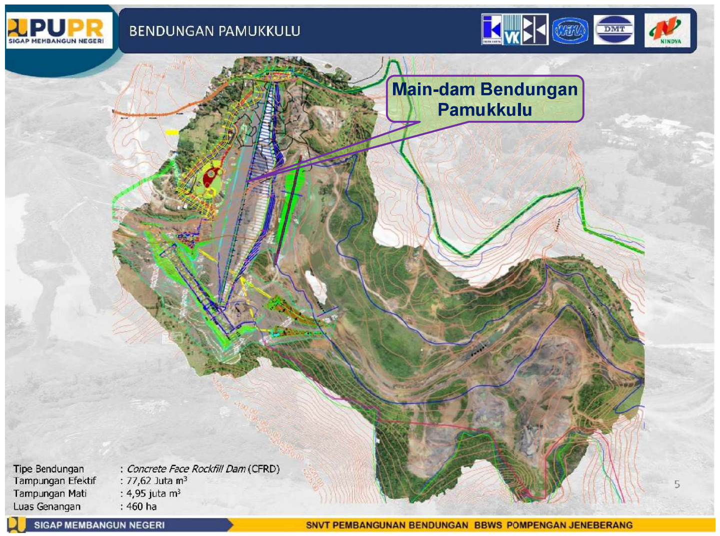
Name of Dam : Pamukkulu Dam
Date : July 19, 2023

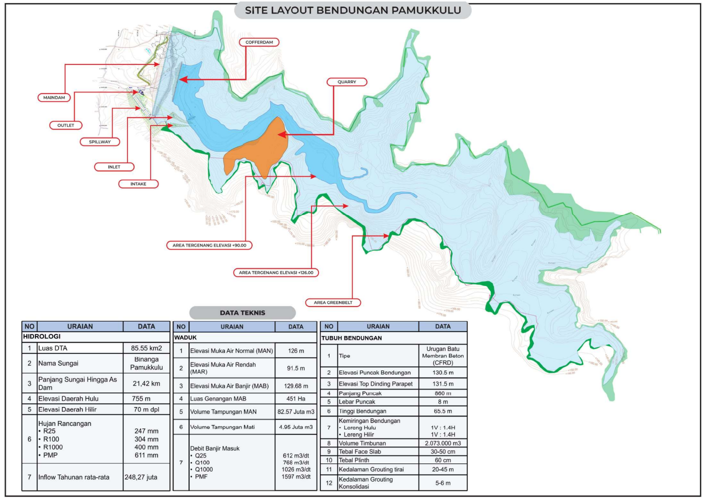
Checklist of Issues

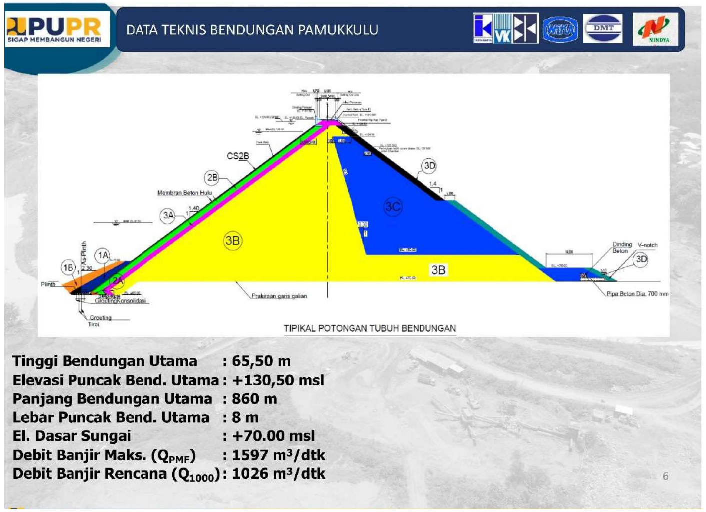
1. General

1.1 Pamukkulu Dam









1.2 Progress/Schedule as of June 2023

No data

Dam embankment works will be started from July and completed at November 2023.

Initial impounding will start within this year.

2. Issues Discussed

In the meeting after site inspection, following two issues are discussed.

2.1 Placing of Concrete Face Slab

At present, face slab on right abutment where the height of face slab is relatively small had been placed as shown in right phot and center portion of face slab, which have longer slab, will be placed from now on. Since in the long face slab concrete placing, it is necessary to pay attention on issues such as consecutive concrete placing works and deformation (shrinkage of concrete).

Comments on face slab concrete placing are as follows:

a. Consecutive concrete placing by slip form should be done so as not to make horizontal construction joints. In some cases, tents are used to prevent the suspension of works due to sudden rainfall.

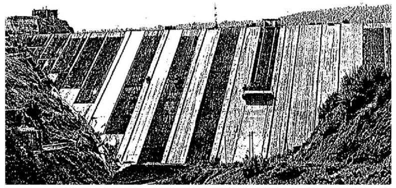
b. In case the work would be suspended, the surface of the concrete should be sufficiently treated (chipping/mortal placing/enforcement by re-bars etc.) so as not to be a weak point.

c. Since the face slab becomes long in the vertical direction and it is expected a large amount of shrinkage, a lubricant such as asphalt emulsion is sometimes used on the bottom surface to avoid constraint of foundation.

d. Slab concrete has already been placed on the right bank abutment, which has a short slab length, but for the long slab that will be placed from now on, the blocks shall be placed and cured alternately at intervals. It is expected to have the effect of adjusting to the deformation of the concrete placed first and increasing the watertightness of both side joints.

e. Curing of slab concrete also shall be done carefully. In general, concrete curing for CFRD face concrete is made two steps as below:

   - Step 1 During Concrete Placing:
     Spray a concrete curing agent (coating curing material) immediately after placing concrete
   - Step 2 After Completion of Concrete Placing:
     Set a watering pipe at the crest of dam, and cure concrete by watering until the impounding.

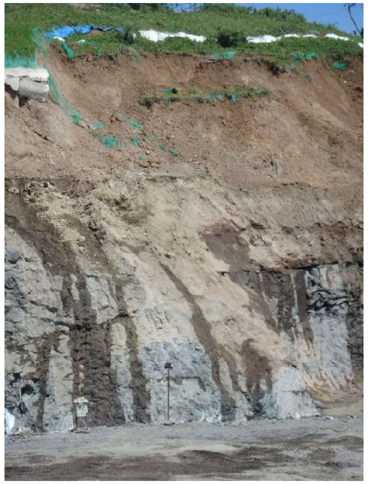
Alternation face slab concrete placing at Ponre-Ponre Dam and Cirata Dam


Blue Sheets on working block against sudden rainfall

2.2 Sliding at Spillway Cut Slope

Strongly weathered rock and weathered residual soil are confirmed just above the fresh bedrock (refer to photos). Groundwater seeps near the boundary with fresh bedrock. This is due to the difference in permeability between weathered and fresh rocks.

The cut slope composed of residual soil of weathered rocks has collapsed on a small scale. In order to protect this slope, it is necessary to remake the cutting slope gentler or to provide slope protection works other than sodding or Geo-net which is proposed by site office because sliding occurred in the soil layer, not slope surface erosion.



Weathered Basalt on Cut Slope and Collapse in the Soil Layer

Site Investigation Report

Hiroshi SHIMIZU
Naoya Mizuno
JICA Dam Construction Advisor

Name of Dam : Ameroro Dam

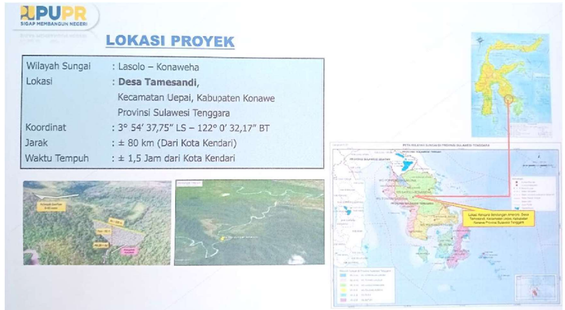
Date : July 20, 2023

1. General

1.1 Ameroro Dam

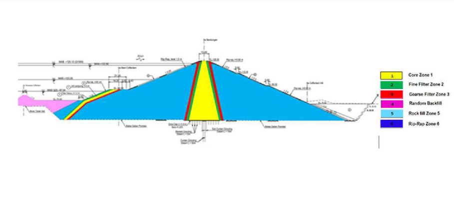
(1) General Information

(a) Location



(b) Dam Features

Dam Name	:	Ameroro
Province	:	Sulawesi Tenggara
Regency	:	Konawe
Coordinate	:	3° 54' 37,75" S 122° 0' 32,17" E



Main Dam		
Type	:	Rock-filled dam with central core
Height	:	82 m
Elevation of crest	:	+128.5 m
Elevation base foundation	:	+ 47 m
Wide of crest	:	12 m
Lenght of crest	:	327 m
Upstream slope	:	1:2.5
Downstream slope	:	1:2.0

RESERVOIR		
Elv. of normal water level (NWL)	:	+122.50 m
Elv. of low water level (LWL)	:	+103.00 m
Elv. of flood water level Q1000	:	'+ – m
Elv. of flood water level QPMF	:	'+ – m
Normal storage capacity	:	88.26 million m3
Dead storage capacity	:	31.71 million m3
Effective storage capacity	:	56.55 million m3

Hidrological Discharge		
5 year return period	:	270.85 m3/sec
10 year return period	:	301.73 m3/sec
25 year old return period	:	340.71 m3/sec
50 year return period	:	369.65 m3/sec
100 year return period	:	398.34 m3/sec
1000 year return period	:	493.21 m3/sec
PMF return period	:	1,222.66 m3/sec

Intake		
Type	:	Drop intakes
Tower Height	:	32.60 m
Tower Peak Elevasion	:	103.00 m
Threshold Elevation	:	+70.4 m

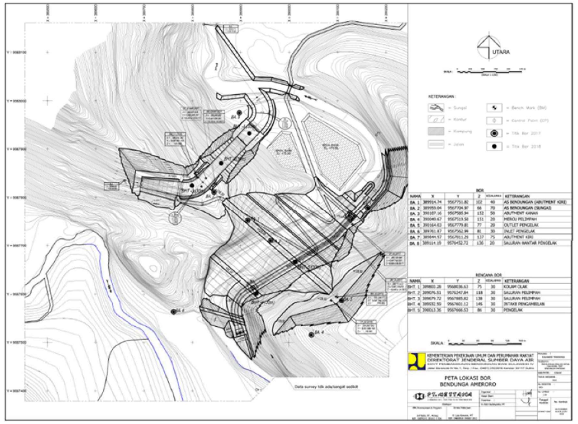
Spillway		
Type	:	Overflow, Ogee
Length	:	329 m
Wide threshold	:	40 m
Crest threshold elevation	:	+122.50 m
Type of basin	:	USBRII

Diversion		
Location	:	Right of main dam
Type	:	conduit
Length	:	380 m
Dimensions	:	4.0 x 4.0 m
U/S elevation	:	+71.0 m
D/S elevation	:	+69.0 m

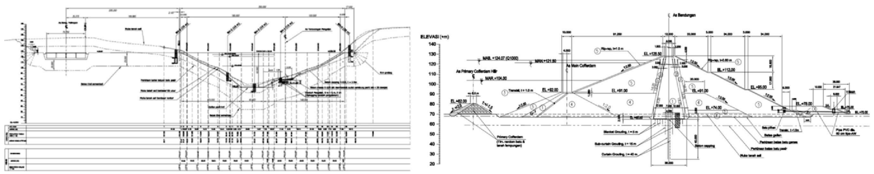
Geology		
Formation	:	Paleozoic Methamorphic
Rock/soil	:	phyllite, slate, gneiss
Rock Class (CRIEPI)	:	CL.CM, CH
Main Dam Foundation	:	Slate
Spillway Foundation	:	Slate
Diversion Foundation	:	Slate

(2) General Drawings

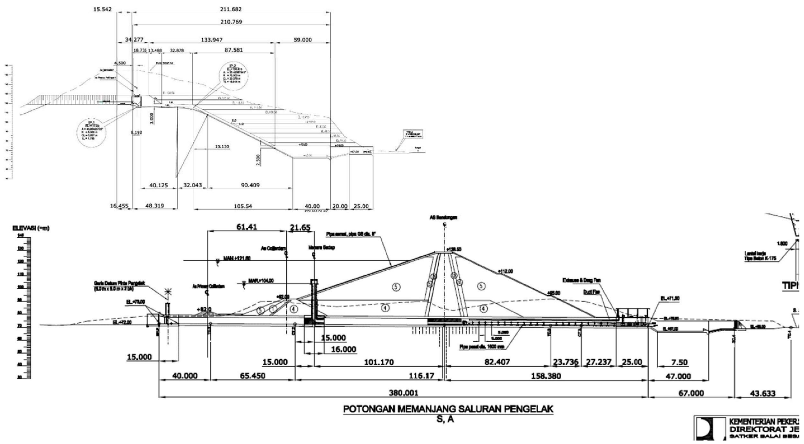
Plan



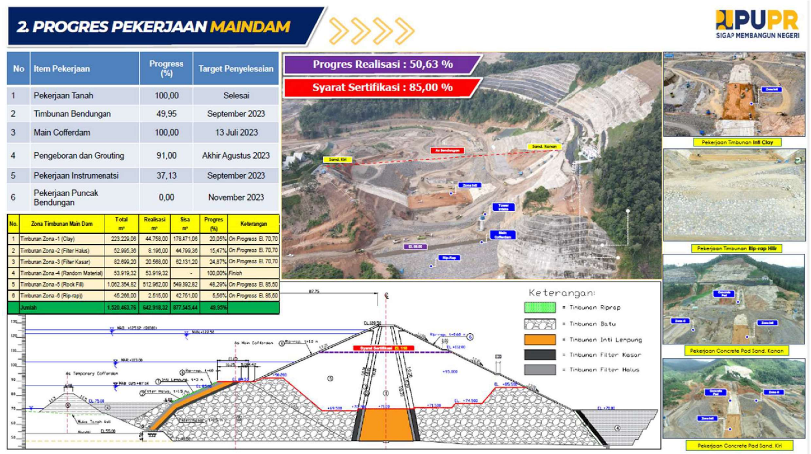
Profile (Upstream View) & Section



Spillway & Outlet Facility



1.2 Progress/Schedule as of July 2023



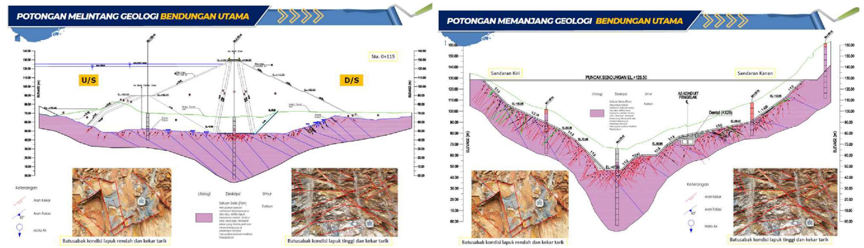
No	Item Pekerjaan	Progress (%)	Target Penyelesaian
1	Pekerjaan Tanah	100,00	Selesai
2	Timbunan Bendungan	49,95	September 2023
3	Main Cofferdam	100,00	13 Juli 2023
4	Pengeboran dan Grouting	91,00	Akhir Agustus 2023
5	Pekerjaan Instrumenatsi	37,13	September 2023
6	Pekerjaan Puncak Bendungan	0,00	November 2023

No.	Zona Timbunan Main Dam	Total m³	Realisasi m³	Sisa m³	Progres (%)	Keterangan
1	Timbunan Zona -1 (Clay)	223.229,06	44.758,00	178.471,06	20,05%	On Progress El. 70,70
2	Timbunan Zona -2 (Filter Halus)	52.995,36	8.196,00	44.799,36	15,47%	On Progress El. 70,70
3	Timbunan Zona -3 (Filter Kasar)	82.699,20	20.568,00	62.131,20	24,87%	On Progress El. 70,70
4	Timbunan Zona -4 (Random Material)	53.919,32	53.919,32	-	100,00%	Finish
5	Timbunan Zona -5 (Rock Fill)	1.062.354,82	512.962,00	549.392,82	48,29%	On Progress El. 85,50
6	Timbunan Zona -6 (Rip-rap))	45.266,00	2.515,00	42.751,00	5,56%	On Progress El. 85,50
	Jumlah	1.520.463,76	642.918,32	877.545,44	49,95%	

1.3 Geological Structure at Damsite

The potential cleavage of the shale strikes a NW-SE (almost parallel to the dam axis) and is inclined downstream (to the east) with an angle of about 20 to 30 degrees.

Besides the cleavage mentioned above, clacks had developed on the excavation slope, with same strike as the cleavage and was inclined downstream with an angle of about 50 to 70 degrees. Weathering has been developed along the cracks. It is supposed that the clacks were formed during the process of upheaving of the ground at this damsite.



2. Issues Discussed

During the site visit, following issues were discussed.

2.1 Slope Stability of Cut-Slope

(1) Cut-Slope of Core Trench on Left Abutment (upstream side)

A small-scale slope failure is observed on the cut slope. As discussed above (Geological Structure), due to the cleavage developing pallarel to the dam axis and tend to weathered and loosen. The weathered shale will be a cause of surface falure and small-scale collapses.

For this reason, many cut slopes are sprayed with mortar.

(2) Cut-Slope at Diversion Tunnel Outlet

Cracks are observed on the shotcrete concrete of the cut-slope right bank of diversion tunnel outlet.

According to the geotechnical engineer at the site, since cracks had occurred, rock bolts (3m) were installed and the deformation of the slope was being monitored. The length of the rock bolts was designed considering the surrounding geological conditions.

(3) Cut Slope at Spillway

Slope stabilization works had been conducted on the cut-slope. Geo-net was installed in the upstream area where fresh rock are distributed, and at the weathered rocks area in the downstream, reinforced concrete slab (15cm thick) are overlayed after placing shotcrete (10cm thick)..

It is supposed that the applied countermeasures are probably appropriate. In case in Japan, however, "Concrete Frame Works with Rock Bolt" is supposed to be applied, judging from the geological condition at site.

Since several small slope failures have been observed at site, continuous monitoring of movement of the cracks and slope surface shall be conducted.

2.2 Filter Materials

Riverbed gravel is used as the coarse filter material, but most of the gravel is flat and oblate, reflecting the geology around this site. The maximum grain size of fine and coarse filter materials are specified as 51 mm and 200 mm, respectively.

The coarse filter materials confirmed at the site were a fairly coarse material with a maximum length of more than 20 cm, and long and oblate. Furthermore, on the dumping and spreading surface, large particles and fine particles were segregated. If the oblate ratio is large, the voids would be large and it may not function as a filter.

When spreading and compacting the filter material, careful work would be required to ensure a uniform grain size gradation.

Fine Filter Material

Coarse Filter Material

3. Additional Information: Slope Failure at Left Cut Slope of Spillway on Sep. 19, 2023

On September 19, 2023, slope failure occurred at left cut-slope of spillway chute.

https://www.youtube.com/watch?v=-pEy0wcFsXA

(1) Comment by Mr. Paulus (Dam Safety Commission):

The landslide occurred on the left side of the spillway chute. It does not harm the existing dam structure at all because the sliding area is only 5m x 10 m.

(2) Explanation by Mr. Anwar (the Consultant):

Proposed countermeasure at present is the "Stretching Construction Method". The works will be conducted slowly and gradually from the top to the bottom by securing it using temporary safety shotcrete.

Site Investigation Report

Hiroshi SHIMIZU
Naoya Mizuno
JICA Dam Construction Advisor

Name of Dam : Temef Dam

Date : July 27, 2023

1. General

1.1 Temef Dam

To be added General Information of Temef Dam

1.1 Progress/Schedule as of July 2023

To be added

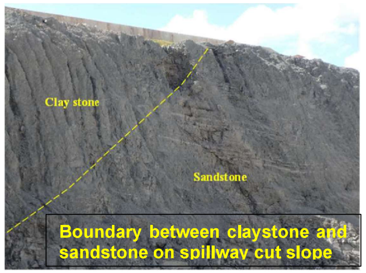
2. Issues Discussed

During the site visit, two issues caused by the claystone were discussed, one is slope-failure at Spillway cut slope and the other is protection of lifting of spillway slab concrete due to swelling pressure.

2.1 General Geology at Damsite

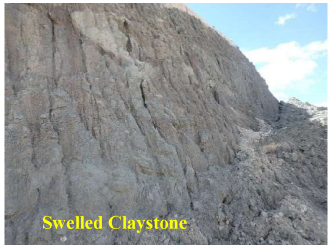
A sedimentary rock layer called "Bobonaro Formation" is distributed aroung the damsite. The stratigraphy at the dam site is sandstone, claystone and limestone layers from the bottom, and the sandstone layer forms the dam foundation.

The strata generally strike in the NE-SW and dip gently to the northwest. The limestone and the underlying claystone and sandstone layers is inconsistent. The claystone layer is divided into two types, namely upper and lower parts, and the upper part, called Scaly clay, is mainly composed of unconsolidated parts. Scaly clay is to be a stratum originally deposited on the seafloor as an olistostrom, which was melanged by tectonic movement during accretion. Scaly clay at the Manikin damsite is consolidated and on the rock fracture surface, the development of cracks with slickensides can be observed.



2.2 Slope Failure at Spillway Cut Slope

Characteristics of Scaly clay distributed at damsite are a) When exposed by excavation, it quickly slakes due to stress release and repeated dry and wet and b) Due to containing swelling clay minerals (montmorillonite), it swells in water. A high swelling pressure (750kPa in max) and swelling strain of 10 to 15% was confirmed by a swelling test.

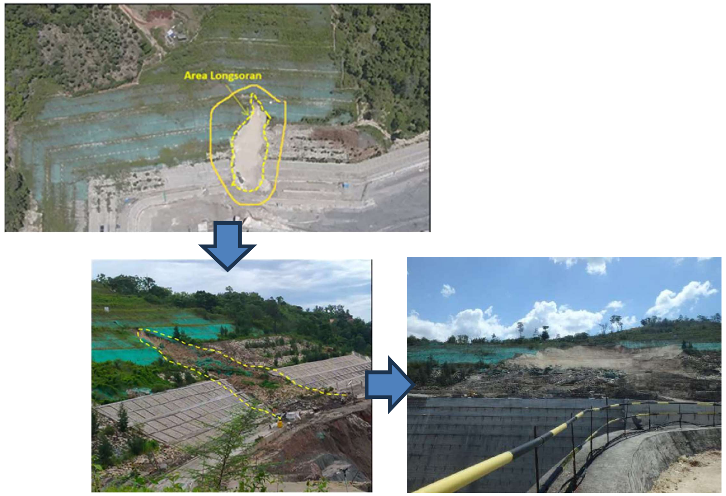


2.2.1 Sliding on Left Bank Slope of Spillway

(1) Present Condition

The collapse is about 100m wide, about 50m long, and up to 7m deep (by eye measurement). The geology of the collapse site is mostly limestone, and scaly clay is distributed near the end of the collapse.

The sliding area has been expanding as shown in photos below.



(2) Couse of Sliding

It is supposed that the sliding occurred due to an insufficient drainage of slope protection works so that rainwater and groundwater accumulated in the limestone layer then the additional weight of water has been leading to the collapse. The limestone layer on the slope is highly permeable due to the development of cracks, and the lower scaly clay is impermeable so these two layers form cap lock structure. It is judged that this cap lock structure is one of factor for the collapse. The fact which the collapse area is topographically located in the center of the ridge, corresponds to the groundwater level generally rises the most at the center of ridge.

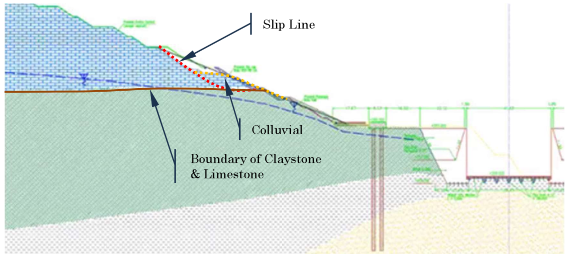
In addition to this geological structure probrem, it is considered that a swelling of claystone also cause of the sliding. Water from permeable limestone layer reached to claystone layer and swelling at claystone surfaice has been accererated. The swelled scaly clay to be functioned as a slip surface.

(3) Proposed Countermeasure

Considering the mechanism of sliding, slope protection shall be drainable measures, such as gabion or riprap.

Currently, two types of slope protection has been constructed at the collapsed area, namely wet stone masonry and riprap work. Since there is no collapse on the slope of the riprap work area, it is applicable to adopt riprap or gabion as a slope protection for this area. Since the toe of sliding is assumed at boundary of limestone and claystone, this portion should be covered with counter weight (riprap or gabion).

When the stability analysis is carried out in design of slope protection works, it is necessary to consider the setting of the groundwater level, especially the groundwater rise during rainy season.



2.2.2 Sliding at Cut Slop of Road

(1) Present Condition

The wet stone masonry with base concrete (B3m x H1.5m) constructed on the gentle slope of about 1:2 has collapsed as if it were peeling off and it is not confirmed that the ground behind the slope is pushing out (refer to photo below).

Scaly clay is observed on the slip surface.

(2) Cause of Sliding

From the conditions of collapse, it is estimated that the scaly clay near the boundary with the protection works swelled and the shear strength decreased, so that it could not withstand the weight of the protection works and then collapsed.

(3) Proposed Countermeasure

However, it was found that a heavy structure would rather cause it to collapse, at site. Therefore, it is appropriate that riprap and gabions, which can not prevent rainwater infiltration but can be expected to have good drainage and conformability to deformation, should be used as slope protection works.

2.2.3 Lifting of Spillway Slab Concrete due to Swelling Pressure

(1) Issues

The foundation at the spillway overflow section is composed of scaly clay. As a result of swelling test of this scaly claystone, a high swelling pressure of up to 750kPa in maximum was observed. Therefore, it is anticipated that the slab concrete will be lifted up under the current design due to the uplift pressure of swelling.

(2) Proposed Countermeasure

Since the swelling pressure is to large, even add the thickness of slab concrete, it is very difficult to correspond the pressure.

Since it is difficult to apply the stress against the swelling pressure by the slab's own weight, it is effective to install rock bolts on the floor to limit the swelling. At that time, the rock bolts are anchored in the sandstone layer distributed below the scaly clay. In addition, before designing, a lockbolt pull-out test shall be carried out to confirm the effect of rock bolt.

In addition, the stability of side wall and overflow weir also checked in case swelling occurs at foundation rock.

Site Investigation Report

Hiroshi SHIMIZU
Naoya Mizuno
JICA Dam Construction Advisor

Name of Dam : Manikin Dam

Date : June 28, 2023

1. General

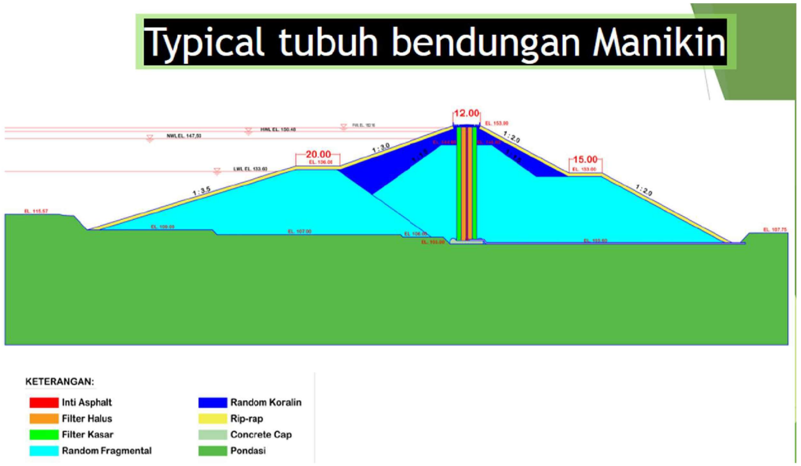
1.1 Manikin Dam

The MANIKIN DAM PROJECT is located between the Kuaklalo and Bokong Village, Taebenu Sub-District, Kupang Regency (14 km from the northeast of Kupang City.

Notable points in dam design are;

✧ Foundation rock is composed of scaly cray stone of Boboranro Formation, which is a rock that easily slake or swell. The results of swelling test show 15-20% of swelling rate and 0.72MPa of swelling pressure. one of the main issues on design of Manikin Dam is to select a proper countermeasure against such deterioration of structural foundation and cut slope.

✧ Considering a limited availability and low quality of soil core materials, the dam type of Manikin Dam has been changed to a random fill dam with asphalt concrete core from a zoned type fill dam with soil core in the original design.



1.1 Progress/Schedule as of July 2023

No data

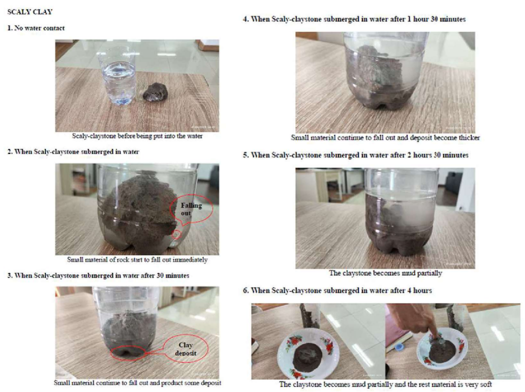
2. Issues Discussed

2.1 Deformation of Tunnel and Tunnel Support

(1) Geological Problems (Swelling of Scaly Cray Stone - Bobonaro Formation)

No spring water was confirmed during tunnel excavation along the scaly claystone, but Olistolith of igneous rock was observed at 160m from the outlet portal and spring water was observed at this portion. In the claystone adjacent to the igneous rock, swelling and slaking occurs immediately after excavation due to stress release by tunnel excavation and water supply from cracks in the igneous rock, and the swelling of the claystone has damaged to the tunnel support.

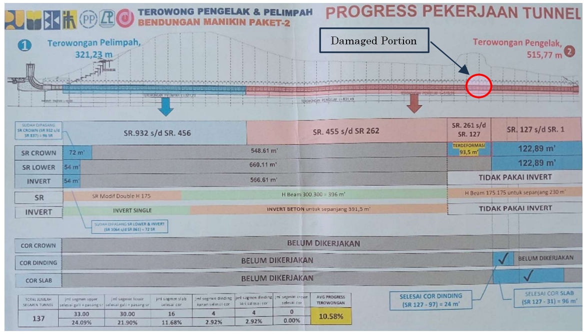
The swelling of the scaly claystone was extremely rapid, and when the sampled rock at the tunnel was soaked in water, it became completely clay in about 4 hours as shown in right figure.



(2) Progress of Tunnel Excavation

At the construction site, the conventional tunnel excavation method using steel rib support and shotcrete has been applied instead of NATM. Progress as of July 2023 is as shown in following figure.

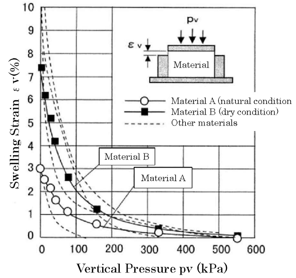
At 160 m from tunnel outlet, damage of tunnel support had occurred then the works has been stopped.



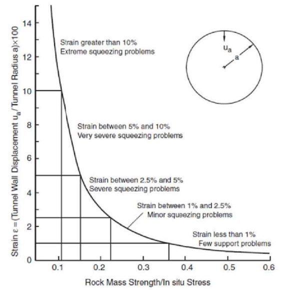
(3) Considerations for Tunnel Support

To prevent swelling of claystone, it is necessary to load a confining pressure to the bedrock that is comparable to the swelling pressure.

The figure on the right shows the relationship between vertical pressure and swelling strain when pressure is applied from above with the side restrained (test results of expansive mudstone in Japan)[1]. As shown in the figure, the amount of swelling strain can be limited by applying pressure to the stress-released surface due to excavation.



In consideration of this relationship between pressure and strain, measures against expansive claystone in the tunnel are usually taken as follows (for reference, degree of squeezing defined by Hoek[2] is shown in right figure).



Tunnelling problems associated with different levels of strain (Hoek, 2001).

【Tunnel Support】

Conventionally, construction methods for expansive (swelling) ground include narrowing the pitch of the steel frame increasing the rigidity or inserting a cushion into the back of the steel frame in an attempt to absorb the expansion has been used.

However, in such a construction method, buckling of steel rib support occurs or re-excavation with new supports are required, so the method of using rock bolts and concrete shotcrete (NATM) has recently applied as a measure to compensate for these issues, and this method is well known to be very effective to the tunnel excavation in the rock to be easily swelled.

[1] A Study on Stress/Deformation Behavior and Stability Evaluation in an Excavated Slope, Nakamura-Yamaguchi-Narita, Dam Engineering Vol. 10, No. 3 (2000)
[2] Hoek, E. Practical Rock Engineering. Chapter 12, 2000 edition

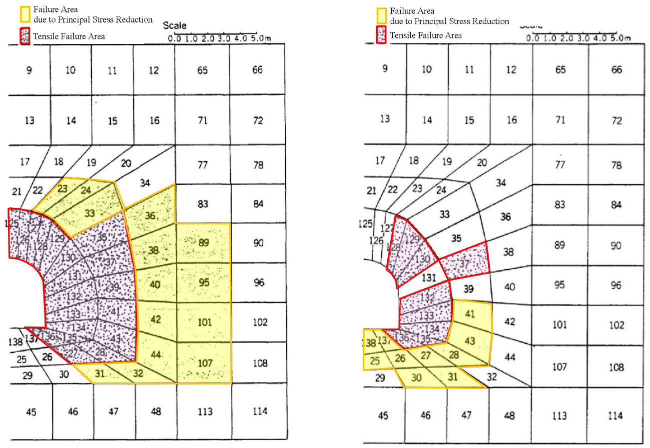
However, NATM cannot apply to bedrocks which are (a) with a large amount of spring water, (b) causes quicksand phenomenon due to spring water, (c) crushed to the point where it is extremely difficult to drill and install rock bolts and (d) where the excavated surface cannot stand on itself. Considering these requirements, it is supposed that the NATM can be applied to the diversion tunnel of Manikin dam.

Prior to design the tunnel support by rock bolt, it is recommended to carry out a pullout test of rock bolt to ensure the effectiveness of rock bolt.

Rock bolt has been input to FEM Model in the following manner:

- The tensile force (compressive force in the ground) acting on the rock bolt acts as a compressive force on the tip and wall of the rock bolt, and the rock bolt is assumed to be a wire rod.

As a result of analysis, the tensile fracture area is smaller than without rock bolts condition, consequently the fracture area due to the decrease in the average principal stress is also smaller.



Without Rock Bolts

With Rock Bolts

Tensile Failure Area and Failure Area due to Principal Stress Reduction

Source: Consideration on Failure Area in Tunnel Excavation in Expansive Claystone, Teramoto-Yoshimura, Nishimatsu Co., Ltd.

In general, tensile stress is generated in the ground around tunnels due to stress release, and tensile failure occurs in this area due to the characteristics of rocks. As a result, the stress shared in this area is not transmitted to other mountains in the vicinity. It is known that the stress found in this way induces new fractures (shear fractures) in the surrounding ground, increasing the loosening area. Therefore, it is extremely important for the stability of the tunnel to prevent tension failure from occurring.

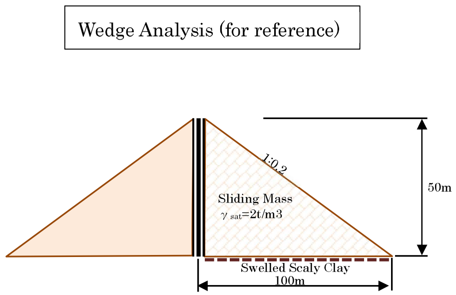
From these facts, it can be judged that the method of using rock bolts and shotcrete as support materials is quite effective.

As discussed above, it is recommended to re-consider the tunnel support system for diversion tunnel of Manikin dam.

2.2 Swelling of Dam Foundation

Temporary excavation for grout cap concrete had already been conducted and the excavated surface was currently completely slaked and deep cracks were observed on the surface, which were considered to be the result of shrinkage of the slaking clay (refer to photos) as well as erosion by rainwater. As shown the present condition, the dam foundation rock consists of scaly claystone easily to swell and slake.

However, the possibility of deformation of grout cap and dam foundation after swelling of this foundation rock and the safety of dam, in particular after impounding, was not discussed at the time of our visit. It is necessary to confirm the safety of dam body and foundation after impounding (after foundation rock is saturated). When the claystone, which is the foundation of the Random material, turns to mud, there is a concern that the strength will suddenly decrease so it is recommended to check the stability of downstream dam body by means of "Wedge method".



Simplified Wedge Analysis
parameters of slaked scaly claystone are;
C = 5 t/m2
φ = 15°
Random material: γ_{sat}=2t/m3
Coefficient of Earthquake k=0.15
Act Force: Water Pressure
W=1/2 h²=1/2 50²=1,250 t
E=100x50x2x0.15 =1,500 t
Resist Force: shearing stress of Clay
C L= 5t/m² x 100m =500 t
σ tan ϕ =1/2x100x50x2.0xtan15=1,340t
Factor of Safety:
FS = (500+1,340)/(1,250+1500) < 1
Note:
1. Physical parameter of clay shall be tested/examined (currently only UU test results are available). Consider the parameters after saturated.
2. Seismic force shall be reexamined.

After careful consideration and provision of the countermeasures, it is necessary to observe the geological conditions in detail during and after completion of final excavation and continuous monitoring of the movement of dam body and foundation before the start of filling and after impounding reservoir.

Reference: Strength of Swelled Claystone

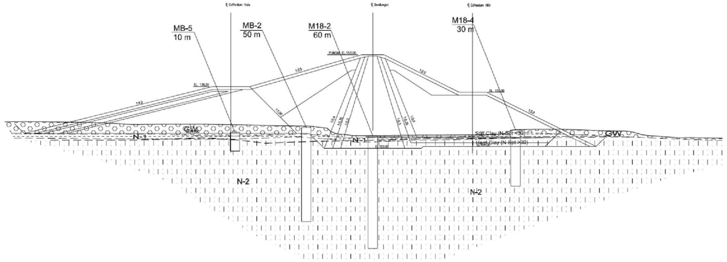
In the design report (Laporan Geologi dan Mekanika Tanah, Sertifikasi Desain Dan Model Test Bendungan Tefmo/Manikin Kabupaten Kepan, PT. Indra Karya, 2018), the reduction of strength of scaly claystone after saturated (refer to following figures). The uniaxial compressive strength of the swelled claystone is only 30% of that of natural condition.

Dam Foundation

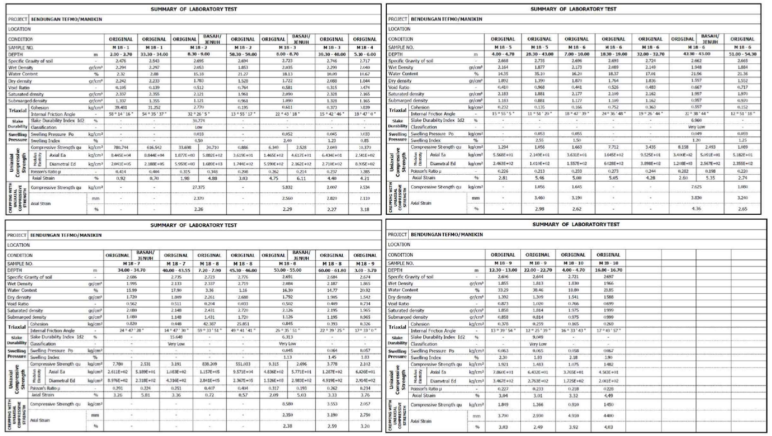
PARAMETER PENGUJIAN LABORATORIUM

Hole No	Depth (m)	Density	Strength qu kg/cm²		Modulus Elasticity			
					Axial Ea (kg/cm²)		Diametral E0 (kg/cm²)	
		gr/cm3	Original	Jenuh	Original	Jenuh	Original	Jenuh
MB18-2	8.3 - 9.0	2.053	33.698	20.710	1876.58	588.16	5958.59	1689.30
MB18-2	58.3 - 59.0	1.853	0.886	-	36.19	-	174.42	-
MB18-4	5.1 - 6.0	2.040	10.370	-	254.08	-	890.47	-

Near riverbed
Already saturated



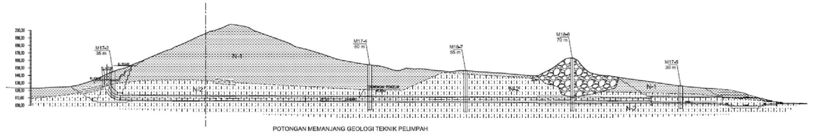
POTONGAN MELINTANG GEOLOGI TEKNIK BENDUNGAN



Diversion Tunnel

PARAMETER PENGUJIAN LABORATORIUM

Hole No	Depth (m)	Density	Strength qu kg/cm²		Modulus Elasticity			
					Axial Ea (kg/cm²)		Diametral E0 (kg/cm²)	
		gr/cm3	Original	Jenuh	Original	Jenuh	Original	Jenuh
MB18-6	7.0 - 10.0	2.173	1.663	-	36.30	-	155.67	-
MB18-6	18.3 - 19.0	2.089	7.712	-	164.54	-	602.80	-
MB18-6	32.0 - 32.7	2.149	3.435	-	95.25	-	389.81	-
MB18-6	42.3 - 43.0	1.948	8.158	2.493	340.03	50.91	1207.92	256.66
MB18-6	51.0 - 54.3	1.884	1.089	-	51.81	-	235.49	-
MB18-7	34.0 - 34.7	1.995	7.780	2.531	261.10	51.88	897.64	231.81
MB18-7	40.0 - 43.55	2.133	3.191	-	108.26	-	431.62	-
MB18-8	7.2 - 7.9	2.337	838.209	-	115722.38	-	284135.64	-
MB18-8	45.3 - 46.0	2.719	551.033	-	95709.11	-	236666.58	-
MB18-8	53.0 - 55.0	2.084	9.315	2.696	483.63	57.71	1526.08	298.29
MB18-8	60.0 - 61.8	2.187	3.778	-	128.72	-	491.93	-



POTONGAN MEMANJANG GEOLOGI TEKNIK PELIMPAH

2.3 Slope Protection

During inspection, slope sliding has been observed on the excavated surface at many locations.

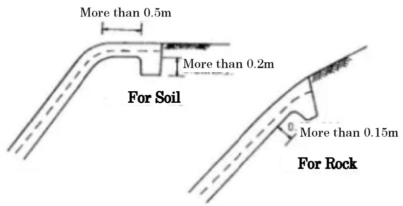
Two types of slope failure were confirmed at the construction site. One is protected with shotcrete after excavation, but collapsed due to swelling due to the infiltration of rainwater, etc. The other is the excavated surface that was left without protection. The surface layer of the slope was deteriorated due to stress releasing and swelling/slaking, and then eroded due to rainwater.

As shown in Photo 1 for the first case, the roadside drainage ditches are provided at a level lower than the road surface, making it easy for rainwater to permeate behind the shotcrete. On the other hands, the deterioration of the spillway excavation slope and the development of gully, shown in Photo 2, are considered to correspond to the latter.

Phato 1 Access Rode on Right Abutment

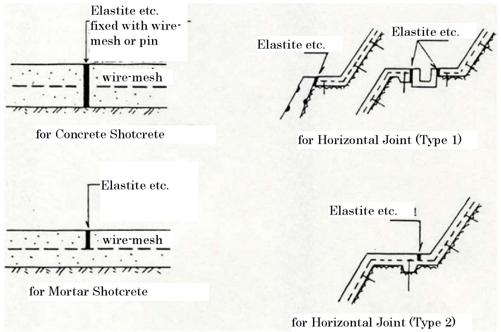
Photo 2 Spillway Cut Slope

In order to protect the excavated slope on swelling claystone, it is essential to install a protection work, such as shotcrete, immediately after excavation and provide a surface drainage system (horizontal and vertical) to the slope surface after shotcrete. It is also important to prevent rainwater from penetrating behind the shotcrete. In particular, in order to prevent rainwater from entering from the ground and stabilize the slope shoulder, cutoff or rounding is provided and the ground is involved to the slope protection (standards design in Japan).



Treatment of Cut Slope Shoulder

Furthermore, it is also possible that the vertical expansion joints are not provided on the shotcrete at the site, it would be a cause of cracks on the surface and increasing the intrusion of rainwater. Expansion joint will be provided as shown in figure below:



Expansion Joint for Shotcrete

"Explanation Joint in Shotcrete"

If the area to be constructed is large and smooth such as cut slope, it is desirable to install vertical expansion joints at a rate of one per 10 to 20m. Also, in principle, it is necessary to provide drainage holes for ground water. As a standard, there should be at least one pipe with an inner diameter of about 4 cm every 2 to 4 m2, and it is necessary to increase the number of pipes depending on the spring water conditions.

It is noted that;

- ✧ Stress release by excavation causes reduction in confining pressure, which in turn results in swelling of soils and the decrease in the shear strength required for slope stability.
- ✧ In the natural ground composed of a soft sedimentary rock like mud stone, which is more or less expansive and/or ready to show weathering, change in soil properties, weakening and softening, due to stress release can be another influential factor for slope instability in addition to the mechanical factors noted above.

2.4 Landslide in Reservoir Area (after impounding)

Surface failure is observed on the slope upstream of the dam site (right photo).

It is anticipated that ground water level will raise up after impounding reservoir and the surface layer of the slope around the reservoir may deteriorate due to swelling/slaking. The slope failure will occur more easily than before impounding and the collapse may expand to the upper part of the slope.

It is necessary, therefore, to survey not only potential area of sliding surrounding reservoir but also existence of objects to be protected by the slope failure and take measures in advance if necessary.

2.5 Foundation Treatment (Curtain Grouting)

(1) Cancelation of Curtain Grouting

Scaly claystone is easy to swell and deteriorated, when water come into the cracks, so that the crack will be closed due to swelling (self-sealing). According to past survey results, the permeability along the dam axis is less than 3Lu. Considering the nature of the foundation rock (claystone), it is expected that amount of cement milk injection is a very little or almost no.

Consequently, it is supposed that the procedure of the curtain grouting may be enough to carry out the regular pilot holes (12m intervals) to confirm the high permeability zone, and then additional holes will be conducted, if necessary.

(2) Rapid Construction

There are several works prior to commencement of the dam embankment work such as;

- ➢ Final excavation of dam foundation,
- ➢ Placing concrete for grout cap/foundation of asphalt concrete core
- ➢ Curtain grouting

The contractor should prepare a method statement for the dam foundation treatment works. When the preparation of method statement, it is essential to consider a dewatering at foundation rock to prevent more damage to bedrock as well as shortening of work time. Furthermore, step implementation of final excavation work to minimize exposing foundation rock for long term.

2.6 Riprap Materials

Due to the limited source of riprap material, the Contractor is proposing to use Exotic rock on left bank (it seems as Olistolith). The rock itself is hard and heavy. It seems good for riprap material. The physical test, however, should be conducted prior to use the rock.

Site Investigation Report

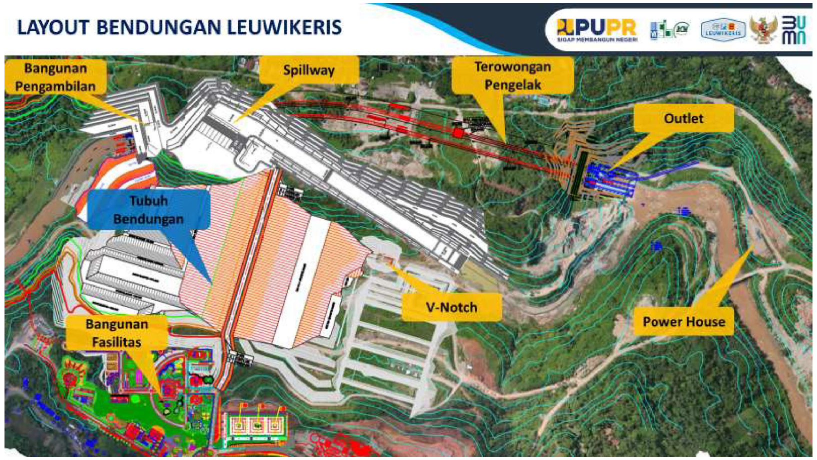
Hiroshi SHIMIZU
JICA Dam Construction Advisor

Name of Dam : Leuwikeris Dam
Date : August 10, 2023

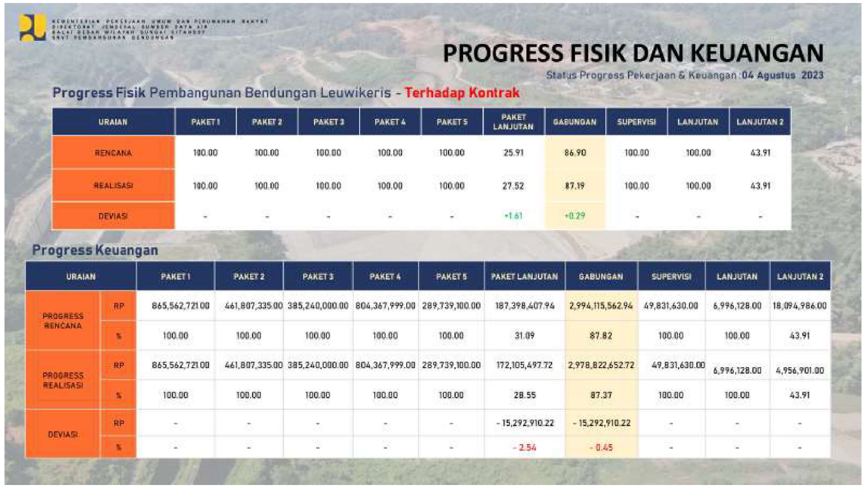
Checklist of Issues

1. General

1.1 Leuwikeris Dam



1.2 Progress/Schedule as of June 2023



PROGRESS FISIK DAN KEUANGAN

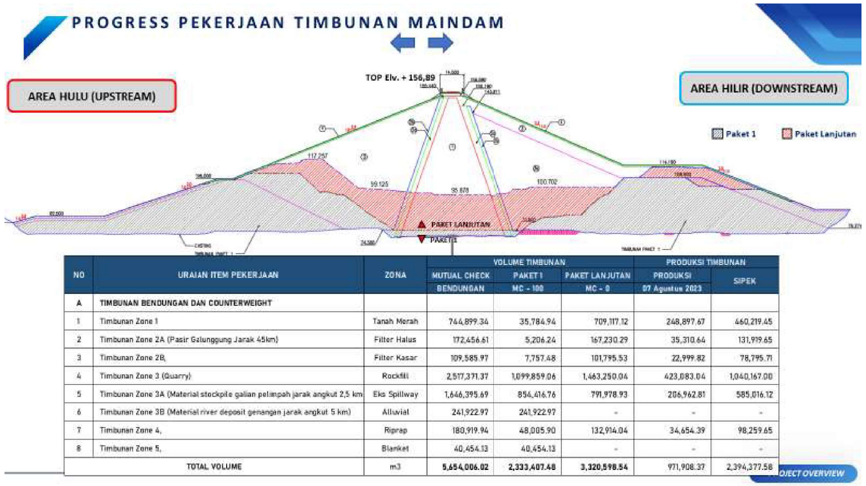
Status Progress Pekerjaan & Keuangan 04 Agustus 2023

Progress Fisik Pembangunan Bendungan Leuwikeris - Terhadap Kontrak

URAIAN	PAKET 1	PAKET 2	PAKET 3	PAKET 4	PAKET 5	PAKET LANJUTAN	GABUNGAN	SUPERVISI	LANJUTAN	LANJUTAN 2
RENCANA	100.00	100.00	100.00	100.00	100.00	25.91	86.90	100.00	100.00	43.91
REALISASI	100.00	100.00	100.00	100.00	100.00	27.52	87.19	100.00	100.00	43.91
DEVIASI	-	-	-	-	-	+1.61	+0.29	-	-	-

Progress Keuangan

URAIAN		PAKET 1	PAKET 2	PAKET 3	PAKET 4	PAKET 5	PAKET LANJUTAN	GABUNGAN	SUPERVISI	LANJUTAN	LANJUTAN 2
PROGRESS RENCANA	RP	865,562,721.00	461,807,335.00	385,240,000.00	804,367,999.00	289,739,100.00	187,398,407.94	2,994,115,562.94	49,831,630.00	6,996,128.00	18,094,986.00
	%	100.00	100.00	100.00	100.00	100.00	31.09	87.82	100.00	100.00	43.91
PROGRESS REALISASI	RP	865,562,721.00	461,807,335.00	385,240,000.00	804,367,999.00	289,739,100.00	172,105,497.72	2,978,822,652.72	49,831,630.00	6,996,128.00	4,956,901.00
	%	100.00	100.00	100.00	100.00	100.00	28.55	87.37	100.00	100.00	43.91
DEVIASI	RP	-	-	-	-	-	- 15,292,910.22	- 15,292,910.22	-	-	-
	%	-	-	-	-	-	- 2.54	- 0.45	-	-	-



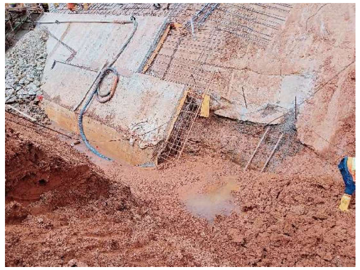
NO	URAIAN ITEM PEKERJAAN	ZONA	VOLUME TIMBUNAN MUTUAL CHECK BENDUNGAN	VOLUME TIMBUNAN PAKET 1 MC - 100	VOLUME TIMBUNAN PAKET LANJUTAN MC - 0	PRODUKSI TIMBUNAN PRODUKSI 07 Agustus 2023	PRODUKSI TIMBUNAN SIPEK
A	TIMBUNAN BENDUNGAN DAN COUNTERWEIGHT						
1	Timbunan Zone 1	Tanah Merah	744,899.34	35,784.94	709,117.12	248,897.67	460,219.45
2	Timbunan Zone 2A (Pasir Galunggung Jarak 45km)	Filter Halus	172,456.61	5,206.24	167,230.29	35,310.64	131,919.65
3	Timbunan Zone 2B	Filter Kasar	109,585.97	7,757.48	101,795.53	22,999.82	78,795.71
4	Timbunan Zone 3 (Quarry)	Rockfill	2,517,371.37	1,099,859.06	1,463,250.04	423,083.04	1,040,167.00
5	Timbunan Zone 3A (Material stockpile galian pelimpah jarak angkut 2,5 km)	Eks Spillway	1,646,395.69	854,416.76	791,978.93	206,962.81	585,016.12
6	Timbunan Zone 3B (Material river deposit genangan jarak angkut 5 km)	Alluvial	241,922.97	241,922.97	-	-	-
7	Timbunan Zone 4	Riprap	180,919.94	48,005.90	132,914.04	34,654.39	98,259.65
8	Timbunan Zone 5	Blanket	40,454.13	40,454.13	-	-	-
	TOTAL VOLUME	m3	5,654,006.02	2,333,407.48	3,320,598.54	971,908.37	2,394,377.58

2. Issues Discussed

Through the site inspection, following 5 issues are pointed out.

2.1 Spring Water Treatment

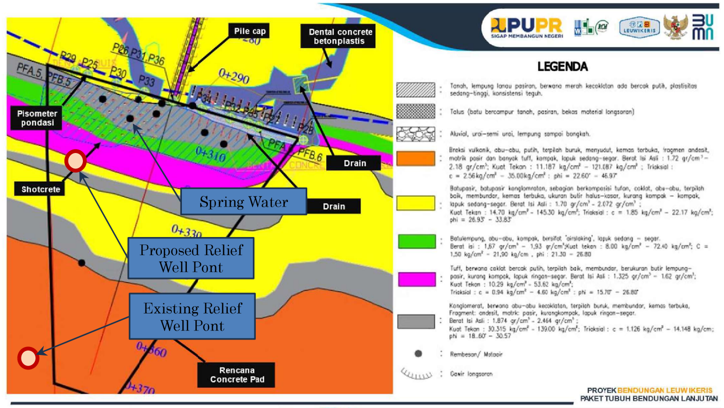
A large amount of groundwater continues to seep at the corner of the right abutment. It is taking time to dispose of it so that the core embankment works have been suspended.

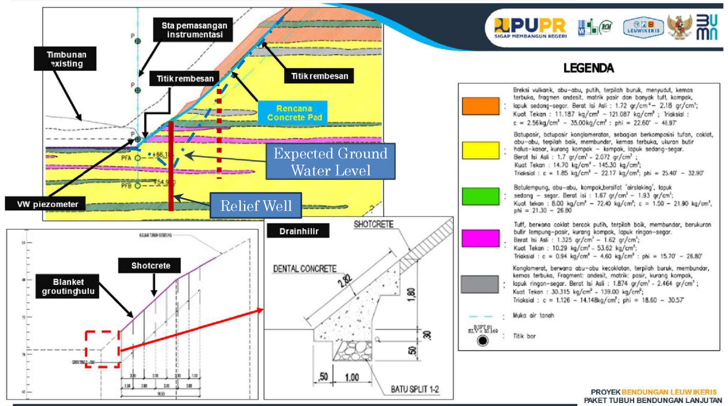


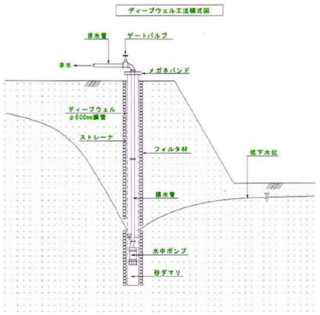
Under the accumulated water condition at the core foundation (filled with water), a compaction of core materials cannot be done optimally, therefore the core embankment site should be maintained as dry condition.

It is already known at site the location of seeping points and source of seeping water (from aquifer), so that the Project office had installed a relief well from the crest of dam to draw down the ground water level. However, it seems too far from the embankment site for the relief well to function effectively and currently the volume of spring water has not been reduced. Considering present condition, it is preferable to install relief well near the embankment site and upstream of aquifer and pump up the ground water to maintain the water level in the relief well lower than dam foundation.

Prior to installing relief well, it is necessary to check the effectiveness of the release well by seepage analysis and at site.







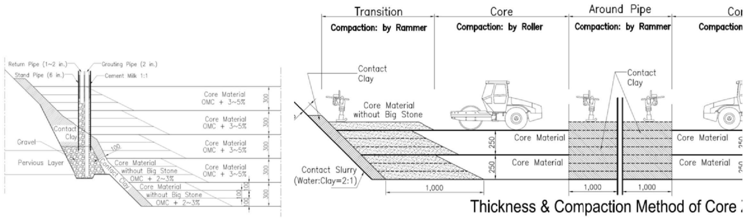
Basic Concept of Relief Well

The other way to treat spring water is to make a drain pit at the location of aquifer and to install stand a pipe and drain spring water by pomp, as shown in photos below. The water level in the stand pipe should keep lower than the top of foundation. Surrounding of stand pipe should be compacted by rummer (refer to following figure).

Spring water

Pit for Drain

Stand Pipe



Spring Water Treatment (example at Jatibarang Dam)

2.2 Random Material

The proposed random material to be applied to the downstream portion of dam body is sandy and easily damaged when exposed to water. Furthermore, it seems to be easily crushed after compacted by vibration roller.

Considering such character of random material, the proper parameter should be examined prior to the stability analysis. For confirming the proper parameter, not only laboratory physical tests but also trial embankment shall be conducted.

2.3 Core Material

Currently, the core embankment works have been conducted, and the following issues have pointed out:

a. The grain size of the materials from Borrow Pit currently used is too fine, and the grain size distribution is included in the "Crack Generation Area" indicated in the USBR standard.

b. The moisture content at the site is about 5% higher than the OMC in particular high at night and in the morning (OMC +7.9%). The compaction, therefore, is not able to perform properly due to the waving phenomenon occurring on the core embankment surface.

If this core embankment continues as it is, it is concerned that the dam body will settle more than designed after completion of the embankment and take time to settle. Furthermore, dissipation of excess pore water pressure in the core will take time, and there is concern that the stability of the dam will be affected due to the residual excess pore pressure.

Currently, the Project Office has proposed a core material blending clay and sand/gravel to address these issues and is discussing the mixing method. They have a plan to mix the two materials at embankment site (damping two materials at site and mixing two materials by backhoe, then compaction). However, this method is very difficult to ensure the quality of core material. There is a risk that the sand and clay will not be mixed uniformly, resulting in uneven particle size and inability to control the OMC of the entire material.

Blending of core materials using stockpile is commonly applied in Japan to keep grain size uniformity and OMC, and is also practiced in many dams in Indonesia such as Bili-Bili dam and Jatibarang dam. An example in Indonesia is shown below.

Blending Method of Core Materials at Jatbarang Dam

- 1) **Stockpiling Method**

  Stockpiling for core material will be executed at the stockyard. The stockpiling method is given as follows:

  - At the stockpile, fine soils and sandy materials with gravel are placed alternately for easy blending. Considering the gradation and the wet density of both materials, the spreading thickness is firstly attempted to be approximately 30 cm.

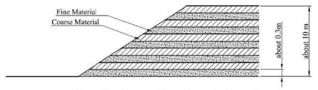


Fig. R 2.2.1 Profile of Layer Structure of Stockpiling (Example)

- Each thickness of fine soils and sandy materials with gravel are adjusted reflecting the laboratory test results during stockpiling so as to achieve the required grading of impervious core material.
- Spreading and leveling of each layer are carried out making the drainage slope by bulldozer.
- The objectionable materials such as grass, roots and oversize stone having a maximum dimension of more than 15 cm are removed to disposal by manpower as much as possible during stockpiling.
- The core material is supplied from core stockpiles where slice cutting is performed by bulldozer, and then, blending and loading is carried out by wheel loader.

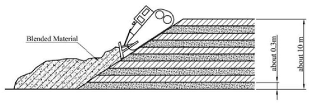


Fig. R 2.2.2 Example of Slice Cutting of Stockpile by Bulldozer

- Moisture content was controlled by adding water or drying to bring it to around 0 % to 3 % higher than the Optimum Moisture Content (OMC + 0~3%).

Photo R 7 Regular Core Stockpiling

Photo R 8 Stockpile Cutting by Bulldozer

Photo R.9 Blending by Backhoe and Bulldozer

Photo R 10 Blended Regular Core Material

2.4 Seepage Measuring Devices

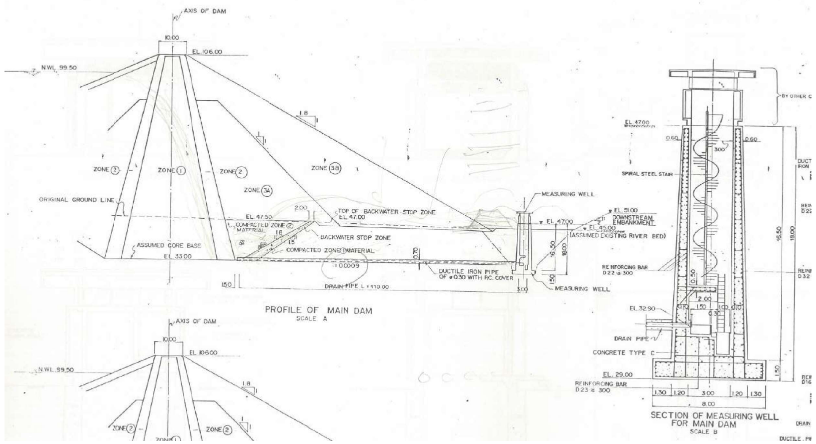
In the original design, outlet elevation of Seepage Measuring Device had been set at lower than river water level so that the counter water from the river will flow into the device and cannot measure the seepage discharge properly. To prevent counter water, V-Notch may be modified in 2 ways namely, 1) raise v-Notch elevation or 2) provide seepage measuring well (tower) for v-Notch.

At present, drainpipes for seepage measuring had been installed and dam embankment is already started, so the second option (Seepage measuring well) may be employed.

Current design of the seepage measuring device will be modified as follows:

- ✧ Set the V-Notch at original elevation,
- ✧ Provide Measuring Well Tower to protect the river water flow into V-Notch, and
- ✧ Install a drain pump which runs automatically when water level of the drain pit (downstream of V-Notch) reach to the setting position.

The capacity of pump shall be set depending on the expected seepage discharge examined by means of seepage water analysis including rainwater on the downstream slope.



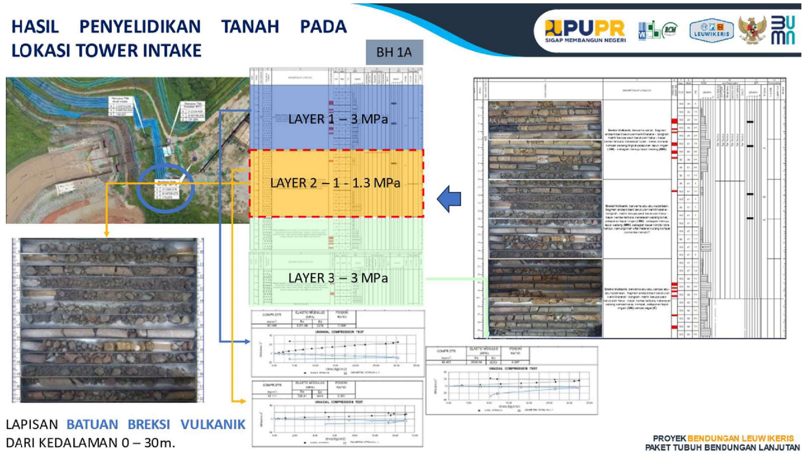
Bili-Bili Dam Seepage Measuring Device (for reference)

2.5 Design of Intake Structure

It was found that a fragile layer intervened in the foundation of the intake tower in the form of a lens during construction of shaft. The Project office changed the location of the tower from just on the tunnel 1 to between tunnel 1 and 2, to avoid the risk of sliding of the tower foundation.

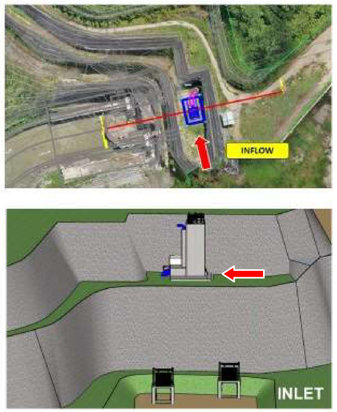
The following advantages are expected through this design change.

- ➢ The tower will be constructed at a distance of 5 m from the cut slope compared to the original design.
- ➢ Since most of the load of tower will be distributed in the original rocks between the diversion tunnels 1 and 2, the load to the tunnel will be minimized.
- ➢ It is anticipated that the hydraulic condition at the entrance of intake tower because the front of inlet is flat. At present, Since the flow in the reservoir is relatively calm, the Project office is judged that there is almost no hydraulic problem.
- ➢ The most of already manufactured hydromechanical components under the original design can be utilized without any modification. Additional member is only elbow pipe connecting to the intake tower.
- ➢ Since the positions of the Intake shaft and Tower are out of alignment, both works can be conducted independently.



Geological Condition at Foundation of Intake Tower

Original Design



Modified Design

Comments

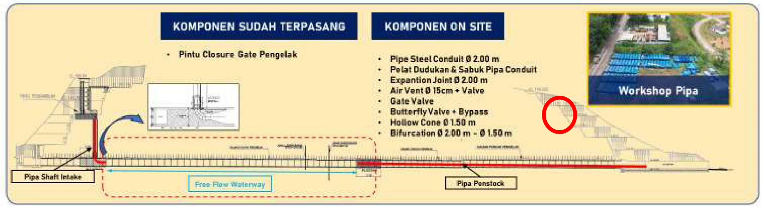
(1) Structural Stability

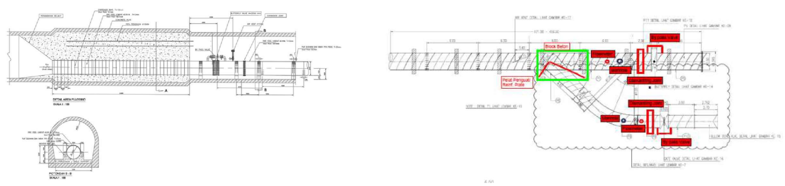
It is important to confirm the distribution of fragile rock mass. Than the stability analysis shall be conducted

(2) Hydraulic Issues

At the site, the consultant explained that the discharge control point of outlet facility was set at the intake facility with control gate (roller gate) and after the control gate, water will flow down through pipe with open flow.

According to the figure below, however, it is supposed that the water flow in the outlet facility is controlled at the outlet with butterfly valve and the roller gate is installed as bulkhead gate for maintenance.





(a) Flow Conditions at Inlet and in Penstock

The flow condition at inlet shall be confirmed with hydraulic analysis (simulation), in particular when the water depth is shallow (check velocity of in front of inlet).

Since the pipe alignment has vertical drop and two bending points with right angle, the flow condition in the pipe shall be confirmed not to occur cavitation or drift flow by controlling the flow velocity.

(b) Discharge Control by Butterfly Valve

In general, butterfly valves can obtain a relatively large flow rate with a small opening, so they are applied for on/off control such as shutoff valves or system operated simultaneously for plural valves, not for flow discharge control. For discharge control, gate valves are utilized, such as jet flow gate, cone sleeve valve and so on.

It is recommended, therefore, to confirm the gate manufacturer the suitability applying butterfly valve for this project before use, in consideration of the operation plan of the valve.

2.6 Others

At the downstream of damsite, where a beautiful rice terrace has distributed, a spoil bank has been developed under the Project. At present, it seems that no works for landscape is planned but it is recommended to provide some measures matching to the successive landscape of rice terrace.

Site Investigation Report

Hiroshi SHIMIZU
JICA Dam Construction Advisor

Name of Dam : Temef Dam
Date : October 05, 2023

1. Temef Dam

1.1 Issues Discussed

The purpose of the site visit is to confirm the conditions of slope failure occurred on September 27, 2023 at left bank cut slope of spillway overflow section and on July 14, 2023 at left bank cut slope of stilling basin.

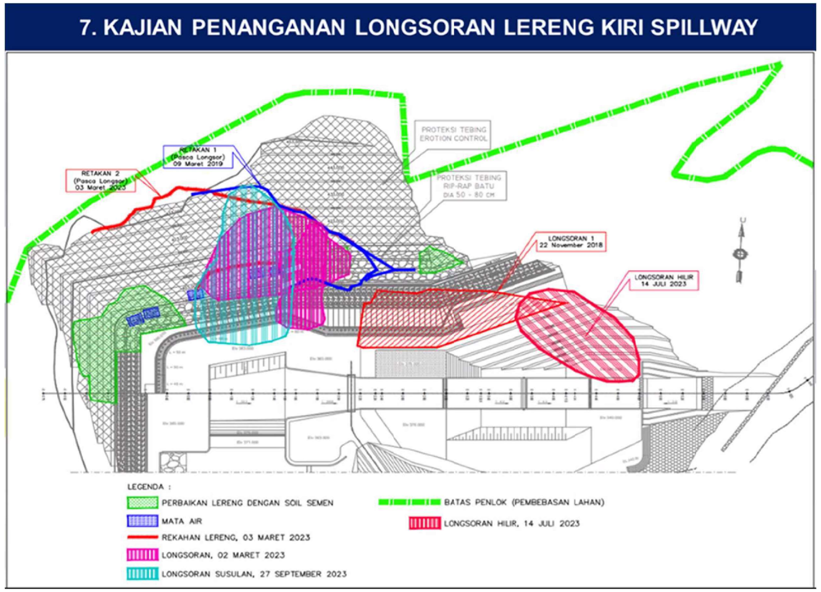
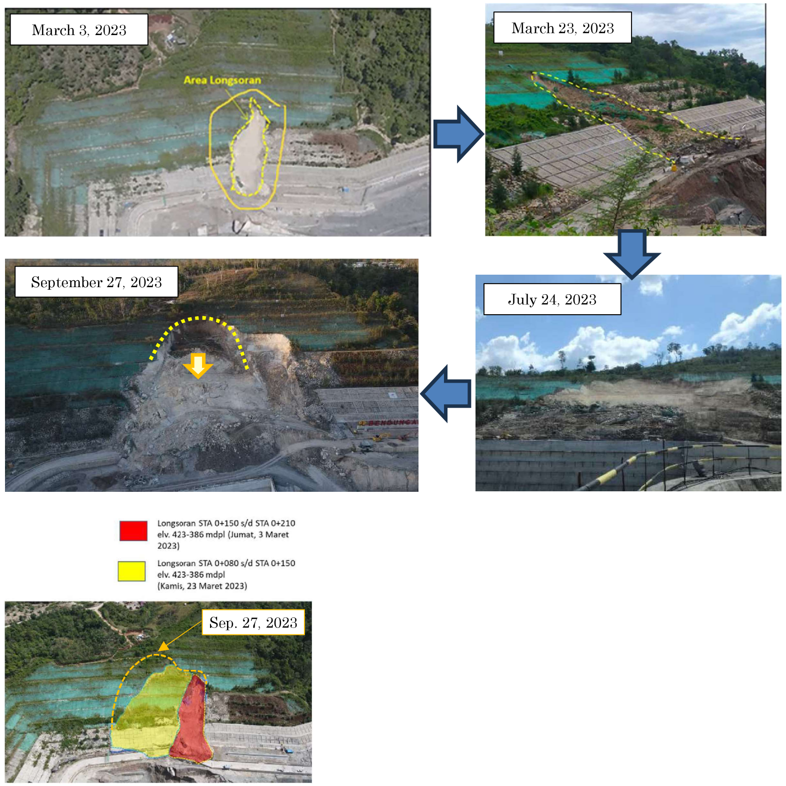


Figure 1 Historical Slope Failure at Left Cut Slope of Spillway, Temef Dam

1.2 Comments on Issues

1.2.1 Sliding at Cut Slope of Spillway Overflow Section

The initial sliding occurred on March 2, 2023 at the left cut slope of spillway overflow section. The sliding has been extended to upper portion and upstream as shown in the above figure and the photos below.

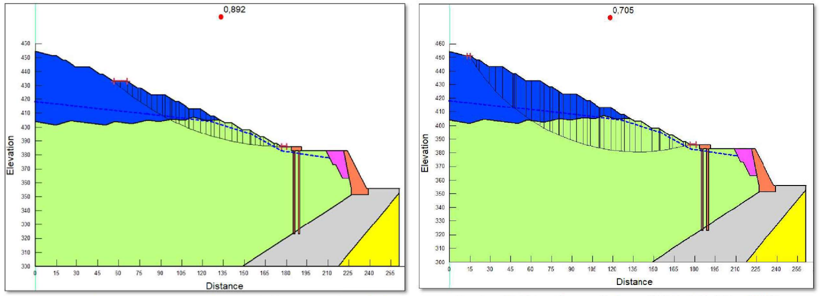


- Sliding area has been developing toward upstream and upward.

(1) Evaluation of Slip-Line

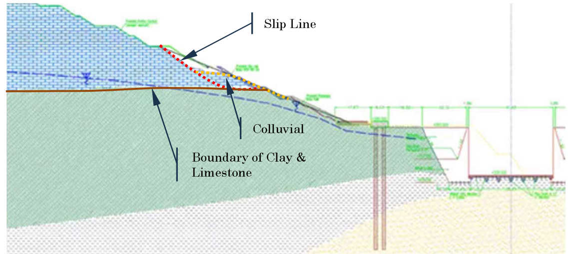
It is supposed that the initial sliding occurred at the boundary of Scaly Claystone and lime stone strata. The ground water is observed along upper portion of Scaly Claystone and due to the excavation for spillway, the stress was released and then swelling of scaly claystone was accelerated at the geological boundary. Since the strength of swelled scaly claystone is very small, it is assumed that the sliding was occurred along the swelled scaly claystone. Furthermore, it can be estimated that the landslide has been gradually expanding to upstream, since the boundary is downward towards upstream.

The Consultant of supervision made an stability analysis applying circular slip method and the estimated slip-line was crossing the geological boundary. However, if the cause of sliding is swelling of scaly claystone at the boundary, the slip-line might be included the boundary (refer to following figure).



Gambar 9. Bidang Longsor Lereng Spillway STA 0+140, (SF = 0,892).
Gambar 10. Bidang Lereng Spillway STA 0+140, (SF = 0,705).

Results of Slope Stability Analysis by the Consultant

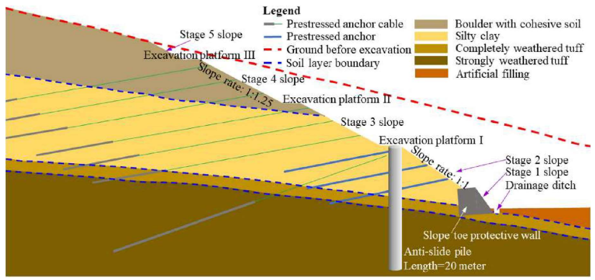


Slip-Line including the Geological Boundary

It is recommended to re-evaluate the slip-line and to analyze the stability of slope along the "Composite Slip-line" including the geological boundary. The thickness of the swelling zone must also be considered during the analysis.

(2) Slope Protection Measures

Considering the mechanism of the sliding, it is important to increase the resist forth at the toe of sliding mass (at geological boundary). It is effective providing a bore piles that penetrating geological boundaries/slip-line at the toe of slip-line or installing ground anchor with concrete wall (or concrete frame) or providing counter weight (embankment) covering the geological boundary.



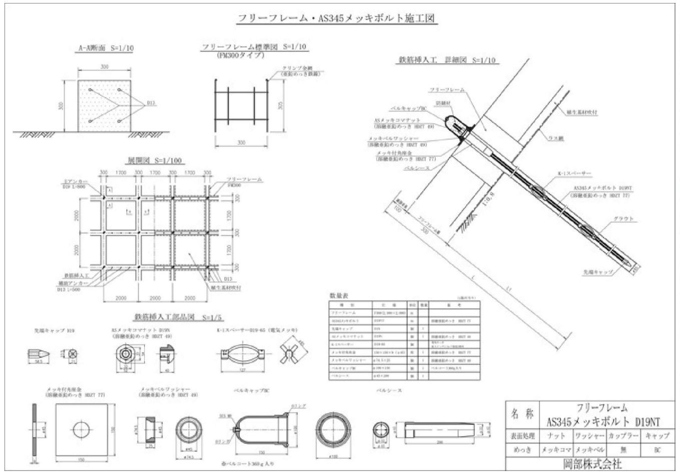
Sliding Protection Works with Bore Pile and Ground Anchor

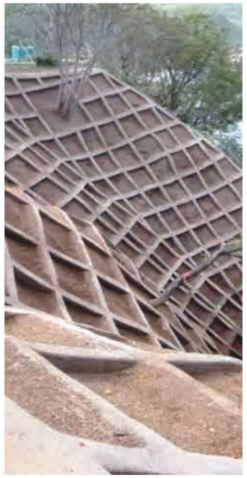
https://www.nature.com/articles/s41598-023-32055-z/figures/3

In addition, the sliding has been expanding at the sliding cliff and in the sliding mass, so that the surface of sliding cliff and sliding mass shall be protected. One of protection measure is Free-Frame with earth anchors (or rock bolts). Frame grid is consisted of steel bars and shotcrete concrete and the frame is anchored with rock bolts, so that the unstable surface can be fixed.

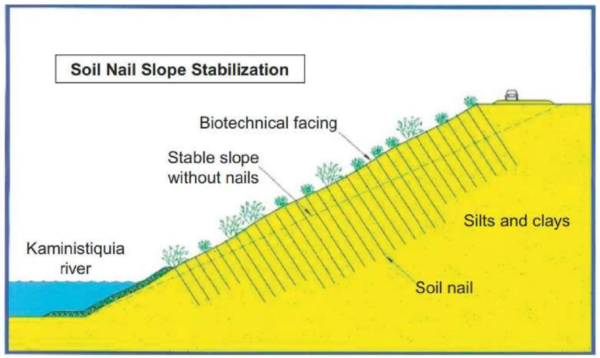
Soil Nailing is also considered to be an effective slope stabilization method.

When selecting a construction method, use it after confirming its effectiveness.





Free Frame with Rockbolts



ıre 21.17 Soil nail and root solution for Kaministiquia riverbank slopes.

Soil Nailing with Geotextile

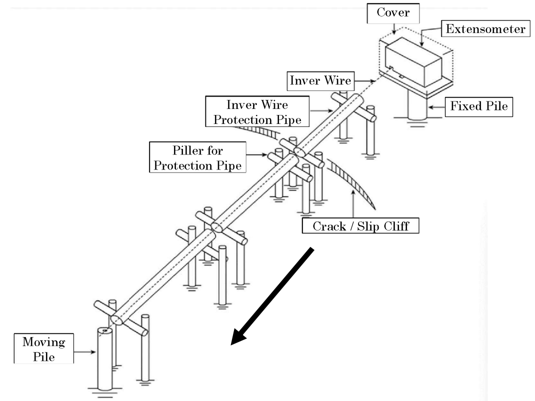
(3) Monitoring of Slope Movement

Ground surface cracks are observed along the top of cut-slope. The movement the crack and sliding surface shall be monitored periodically. Monitoring shall be conducted as below:

(a) Extensometer

When landslide activity increases and cracks occur on the ground surface, extensometers shall be installed at the cracks in order to monitor the expansion of the cracks and the movement of the landslide periodically. It is preferable to introduce a system that can take continuous automatic monitoring in consideration of sudden changes.

The standard extensometer is shown in figure below:



Extensometer for Monitoring of Landslide

(b) Fixed Point Observation

In order to monitor the displacement of the ground surface (sliding mass) due to landslides and surface collapses, we will periodically conduct fixed-point observations by installing Movement Monuments, reflective prisms, or reflective sheets at observation points.

1.2.2 Slope Failure at Cut Slope of Stilling Basin

It is estimated that the character of the sliding at the cut slope of stilling basin is surface sliding. The sliding occurred at the weathered (slaked) surface of alternation of sandstone and claystone layer due to the release of load by excavation.

Therefore, it is recommended that protection works at this location is soil nailing and slope surface will be covered with geotextile.

The area of protection works shall cover the whole slope surface consisting of the claystone layer.

1.3 Manikin Dam

1.3.1 Protection of Deformation of Tunnel Lining Concrete

Currently, the contractor is conducting a stress analysis that takes into account swelling pressure in the tunnel lining concrete.

Since the main load on the tunnel lining is swelling pressure, it is important to minimize the occurrence of swelling in scaly claystone. In order to prevent further swelling of the scaly claystone, the concrete lining must have a structure that can withstand reaction forces greater than the expansion pressure. It is possible to reduce the burden on the concrete lining with anchor bolts and rock bolts.

1.3.2 Monitoring of Tunnel

In general, monitoring of tunnel deformation should begin immediately after excavation. Monitoring items include measuring not only the deformation of the tunnel interior, but also the stress in the shotcrete, lining concrete, and steel rib supports.

For details on tunnel monitoring methods, following paper and etc. can be referred.

ITA Report n 009 - Monitoring and control in tunnel construction - N ISBN: 978-2-9700776-3-3 / NOV 2011

https://about.ita-aites.org/publications/wg-publications/65/monitoring-and-control-in-tunnel-construction

1.3.3 Reinforcing Bars Arrangement

In the tunnel, reinforced bars arrangement for lining concrete already commenced. Due to the large stress occurred in the lining concrete, the re-bars are provided very dense (The space between re-bars are CTC100 cm). Since the size of the re-bars are D22 to D25, the free space between re-bars is only 5cm.

The free space between re-bars is specified for the following reasons.

a. Aggregates can clog between re-bars under an inadecuate space, resulting in insufficient concrete filling and creating cavities within the concrete.

b. If the spacing between re-bars is narrow, stress cannot be distributed appropriately to concrete.

Generally, the free space between reinforcing bars are specified the largest value among:

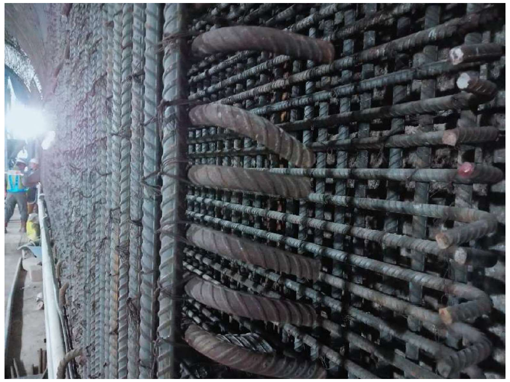
a. 1.5 times of re-bars diameter + outermost diameter of re-bar,

b. 1.25 times the maximum size of coarse aggregate + outermost diameter of re-bar, and

c. 25 mm + outermost diameter.

In addition, even if the minimum spacing between re-bars is secured, there is a risk that the concrete will not be able to fully penetrate between the reinforcing bars, so it is important to compact the concrete by vibrater thoroughly when pouring it.

It is recommended to check the re-bar arrangement and try to secure the free space between re-bars as much as possible.



Site Inspection Report

Hiroshi SHIMIZU
Naoya MIZUNO
JICA Dam Construction Advisor

Name of Dam : Bagong Dam
Date : October 11-13, 2023

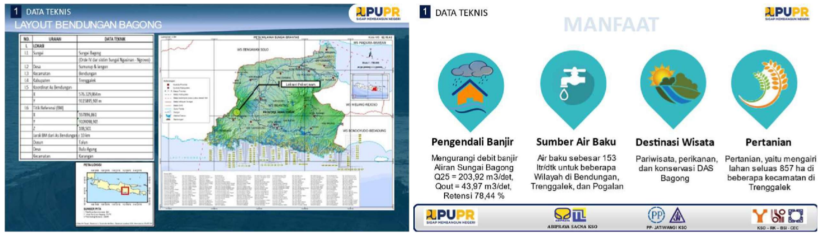
1. General

1.1 Bagong Dam

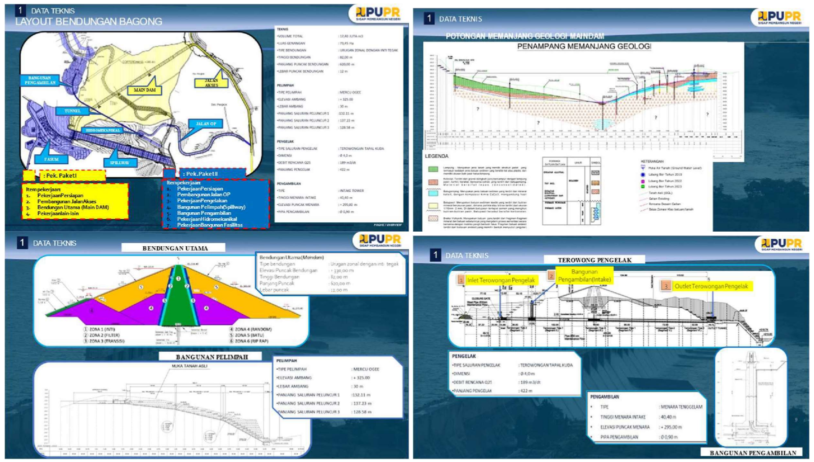
The Bagong Dam is currently being constructed on the Bagong River, branch of the Brantas river, Trenggalek Regency, East Java Province.

The type of dam is center core type rock fill dam. The height of the dam is 82 m, and the crest length is 620 m, with a total storage volume of 20.49 million m^3. The main function of the Bagong Dam is municipal water supply (465 ltr/s of raw water to Trenggalek city) and irrigation water supply (for 857 Ha of Bagong irrigation), as well as flood control (reduction of flood discharge (25-year flood: from Q_{in} = 203.92m3/s to Q_{out}=43.97, reduction rate 78.44%).

(1) Location and Purpose of Bagong Dam



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



2. Progress/Schedule as of June 2023

[to be confirmed]

3. Issues Discussed

During the site inspection, following three (3) issues were Discussed:

(a) Stability and permeability of right abutment and spillway foundation
(b) Treatment of thick colluvium at left abutment
(c) Collapse of excavation face at tunnel crown

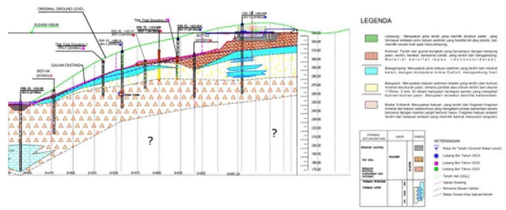
Regarding (a) and (b), in particular, it was found that a thick layer of colluvium was distributed at both abutments of the dam foundation and the properties of the colluvium on the right and left banks are different, after the topsoil was removed. The colluvium at left abutment is classified as sliding debris but the colluvium on the right abutment can be classified a very loose rock mass, the origin of which is estimated to be creep. This fact was considerably different from the geological interpretation in the dam design stage.

The detail description of the geology is shown in Attachment 1.

3.1 Right Abutment of Dam and Spillway Foundation

3.1.1 Geology

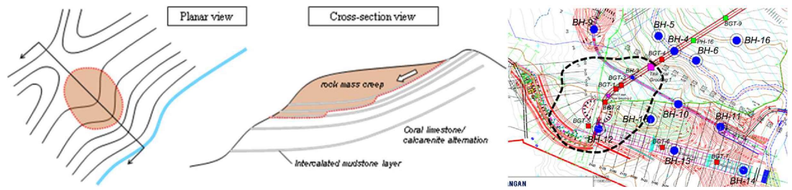
The retention of geological structures within the colluvium suggests that the colluvium has formed by crept or moved as a mass. The slope is close to the top of ridge, indicating that the distance over which the mass has moved is short.



Geological Profile of Spillway

Based on the material provided for the site inspection and interviews with the site engineer, the distribution area of the colluvium is estimated as shown in following figure.

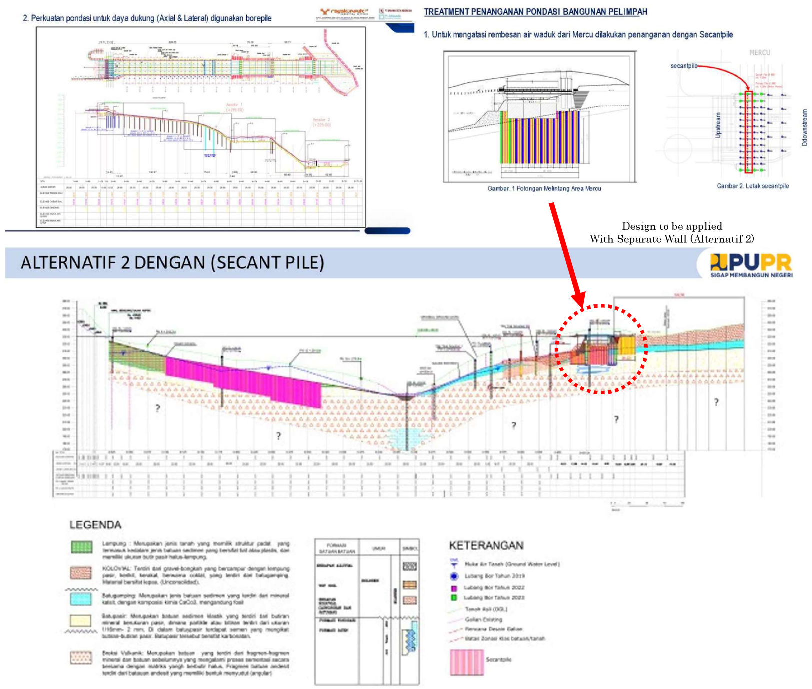
Judging from the facts of (a) no landforms suggestive of land sliding (e.g. slip cliffs, gentle gradients at the head) around the right bank slope (by reading of Google Earth satellite image), (b) characters of colluvium (eg. distribution of open cracks, weathering condition and remaining of geological structure in the sliding mass) and (c) the short distance of movement, the colluvium is evaluated to be rock mass creep.



Schematic Diagram of Rock Creep on the Right Bank

3.1.2 Currently Adapted Design against Foundation on Colluvium (Alternatif 2 Dengan (Secant Pile))

In the currently adapted design, the colluvium layer remains as foundations of spillway and right abutment of dam. As a countermeasure for spillway foundation, a Secant Pile (bentonite-cement wall) as a continuous underground wall is applied to reduce seepage flow and to prevent piping, besides the concrete bore pile for the spillway (see figures below). To secure a water tightness in the colluvium, a continuous underground wall has been proposed applying Secant Piles using very deformable bentonite-cement.



3.1.3 Comments and Recommendations

(1) Comments

- ✓ Considering that the concerned foundation can be evaluated as rock creep, there are risks such as (1) existence of cavity in the foundation, and (2) smaller strength along the slip line than the value used in the stability analysis.
- ✓ In the long term, there is a possibility that the main dam foundation will be unstable since the wedge-shaped slope remaining between the spillway.
- ✓ In the current design, a Secant Pile, using bentonite-cement as a plastic material (see table below) is applied to secure the water tightness along the dam axis, besides concrete bore pile will be used for the spillway.
- ✓ When the colluvium layer (or creep layer) of the foundation deforms due to an earthquake, etc., a gap may occur between the dam body on the bedrock (colluvium) and the spillway structure with concrete pile foundation. A risk of inequality deformation, therefore, will occur at the joints of these structures' foundations and the gap will be a water path.
- ✓ Generally, to avoid this risk, pile foundation is not employed to dam structures that are subject to water pressure.

Note 1 : The design strength of the colluvium is c=24.5kPa and φ=16°, regardless of whether it is on the left or right bank. This value generally corresponds to the strength of a soft clay layer. However, since shear planes develop in the clay of the slip layer, the internal friction angle may be less than 10°.

Note 2 : Characteristics of Materials for Underground Wall (for bulkhead)

Jenis Material	Kekuatan (Mpa)	Permeabilitas (m/dtk)	Deformabilitas	Sifat Retakan	Komposisi
Beton tremi	30 – 50	10^{-12}	Kaku	Mudah retak (brittle)	Semen + agregat slump 20 cm w/c = 0,5 – 0,7
Beton plastis	1,0 – 3,0	10^{-9}	Tinggi	Tahan terhadap regangan sampai 10 % tanpa retak	Semen + agregat + lempung + bentonit w/c ≥ 0,5
Slurry semen/ bentonite	0,1 – 1,0	10^{-8}	Sangat tinggi	Tahan terhadap deformasi dan tekanan lateral	Semen + bentonit w/c ≥ 0,5

(2) Recommendation

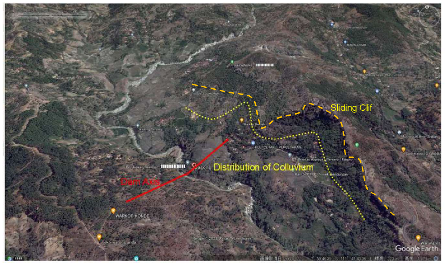
- ✓ A detailed geological survey should be carried out again, to confirm the distribution/character of colluvial and the existence and shape of the slip layer, then the design should be modified in accordance with the actual conditions.
- ✓ Since a slip layer exists in the creeped rock generally, it is necessary to carry out a stability analysis using the underside of the colluvium as the slip surface, besides the arc-slip analysis.
- ✓ Since it is important to confirm the strength and shape of the slip layer for the stability analysis, the results of the existing geological investigations should be reexamined from the above perspective, and additional investigations should be carried out if necessary.
- ✓ It is necessary to confirm the final settlement of colluvium layer after embankment, in case the colluvium layer would be remained as dam foundation.
- ✓ It is recommended to avoid piles foundation and try to use spread foundations for spillway. In case concrete volume becomes large amount, the foundation can be replaced by means of cemented material such as CSG and soil cement.
- ✓ The required covering area of Secant-pile, especially at the rim of dam body and spillway shall be reexamined through the seepage analysis.

Note 1 : Detail evaluation and comments on the Alternative 2 design which is currently proposed is attached in Appendix 2.

Note 2 : Applied measure to the rehabilitation of Oroville dam Spillway is presented in the following IRL: https://www.youtube.com/watch?v=ekUROM87vTA.

4. Colluvium at Left Abutment

4.1.1 Site Condition



According to Google Earth satellite imagery, steep slopes or cliffs can be seen above the left bank slope (right photo). It is estimated that the sediment generated by the slope failure near the summit has repeatedly moved and accumulated due to colluvium, forming a thick layer of colluvium below the slope, and the area where colluvium is distributed forms a gentle slope.

4.1.2 Curent Design and Proposed Measure

In the original design, the core foundation is proposed on the colluvium layer.

Since the bearing capacity of the colluvium and stability of the foundation have been confirmed by the consultant, the challenge is to secure the water tightness in the colluvium as dam foundation.

To secure the water tightness in the foundation, an underground continuous wall using Secant Pile is proposed. The secant pile consists of a very deformable plastic material made of bentonite and cement (compressive strength 2-3 MPa).

4.1.3 Recommendation

- The disparity of settlement between colluvium layer and Secant Pile after dam embankment shall be confirmed.
- The required covering area of Secant-pile, especially rim of dam body shall be reexamined through the seepage analysis.

5. Rock Collapse at Tunnel Crown

(1) Present Condition

Claystone layer have been confirmed at the tunnel outlet and are distributed at an upward angle with few degrees toward the upstream or horizontal. It is confirmed at site that the claystone has developed shear planes and is fragile. This claystone layer (or the upper claystone layer) is currently appeared at the crown of the tunnel excavation face. During the tunnel excavation at this section, the rock mass below the claystone had been fallen down and It is dangerous to continue the works. Since it is anticipated that the claystone layer will continue for several sections, the discussion on this matter is focused in measures to ensure safety during excavation.

(2) Proposed Countermeasure

Until the rock mass at crown would be stable, one cycle of excavation length (blasting length and the interval of steel rib support) should be shortened (for example, about 50 cm per cycle). Immediately after installing the steel rib support, concrete shall be placed by shotcrete to fill up the gap between the steel rib support and the excavation surface, and then, rock bolts will be installed. At this time, the spacing and length of the rock bolts at the crown where has a risk of collapse, shall be reviewed in accordance with the excavation cycle.

It is also necessary to increase the number of blasting holes and reduce the amount of blasting charge used per hole, to minimize damage to the surrounding.

Fore-poling is available as an auxiliary construction method. However, since the angle of the claystone layer is gentle (only few degrees to upward or almost horizontal), when using it, it is important to confirm the geological structure before deciding on the insertion angle.

6. Conclusion

Since the actual geological structure of the damsite is considerably different from the results of geological analysis at the design stage, it is essential to carry out the geological survey again and to verify the origin, formation, distribution, and characteristics of both banks colluvium.

After verify the geological conditions at the Bagong Damsite, the applicability of current design must be checked based on the newly obtained geological information.

Dams are huge structures, and if they breach, they would be a cause of tremendous damage. Therefore, when issues of unknowns are contained, it is necessary to seek countermeasures that have the advantage of least amounts of risks and fewest unknowns, and to carry out design and construction paying highest attention.