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

The Project for Capacity Development of Mt. Semeru Volcanic Disaster Structural Measure Planning in The Republic of Indonesia

Project Completion Report
Annex (7) Final Report
Appendix Volume 1

Feburuary 2025

Japan International Cooperation Agency (JICA)

Yachiyo Engineering Co., Ltd.

Appendix

Appendix 4-1　Applied Japanese Technical Standards

Outline of the Applied Japanese Standard on Design

Title of Japanese standard	Contents	Application Conditions
Technical Guideline for Establishing Sabo Master Plan against Debris Flow and Driftwood, National Institute for Land and Infrastructure Management, Japan, 2016 砂防基本計画策定指針（土石流・流木対策編）解説、国土交通省国土技術政策総合研究所、2016 年	Section 1. Overview Section 2. Basic items in debris flow and driftwood countermeasure planning 2.1 Basic guideline to planning 2.2 Objects of protection 2.3 Design scale 2.4 Reference point 2.5 Design volumes of sediment and driftwood 2.6 Method to survey and calculate sediment and driftwood volumes Section 3. Debris flow and driftwood control plan 3.1 Basics of enacting debris flow and driftwood control plan 3.2 Design capturing volume of debris flow and driftwood 3.3 Design depositing volume of debris flow and driftwood 3.4 Design volume of debris flow and driftwood prevention Section 4. Spatial distribution plan of facilities against debris flow and driftwood 4.1 General principles 4.2 Principles of spatial distribution plan of facilities against debris flow and driftwood	The latest technical standard for the design of Indonesian Sabo Facilities is based on "SNI 2851-2021 Design of Sabodam, 2021." When designing Sabo Facilities, it is recommended to use this standard as the primary reference. For items not covered by this standard, it may be possible to refer to previous Indonesian technical standards and guidelines. Any content not covered by Indonesian standards can be supplemented with the Japanese standards listed on the left. It is recommended to thoroughly review the relevant Japanese standards and use them as a reference for the design. The

	4.3 Functions and locations of debris flow and driftwood countermeasure facilities Section 5. Sediment and driftwood removal	English translations of the relevant Japanese standard is attached on the following page as a reference document.
Technical Guideline for Designing Sabo Facilities against Debris Flow and Driftwood, National Institute for Land and Infrastructure Management, Japan, 2016 土石流・流木対策設計技術指針解説、国土交通省国土技術政策総合研究所、2016 年	Section 1. General Principles Section 2. Design of debris flow and driftwood facilities 2.1 Debris flow and driftwood capturing work 2.2 Debris flow and driftwood restraint work 2.3 Debris flow torrent training work 2.4 Debris flow depositing area 2.5 Erosion control greenbelt 2.6 Debris flow direction-controlling work Section 3. Sediment and driftwood removal Section 4. Setting design external forces during a debris flow 4.1 Calculation of design external forces during a debris flow 4.2 Impact load of boulders 4.3 Impact load of driftwood	Same as above
Part 7 Construction, Sabo Facility Design Guidelines Chubu Regional Bureau, Japan, 2020	Chapter 1: Temporary Works 1. Diversion Works 1-1. Target Flow 1-2. Temporary Cofferdam Works 1-3. Temporary Drainage Channel Works 2. Water Division Works	There are currently no Indonesian standard/guideline regarding the construction methods for Sabo facilities. The Japanese standard referenced in this project, "Part 7:

砂防施設設計要領 第 7 編 施工、国土交通省 中部地方整備局、2020 年	3. Construction Roads 3-1. Forest Road Regulations 3-2. Construction Road Design 3-3. Installation of Safety Facilities 4. Temporary Bridge Construction 5. Temporary Facilities Chapter 2: Concrete Pouring Plan 1. Lift Height 2. Concrete Pouring Sequence 3. Considerations for Pouring	Construction, Sabo Facility Design Guidelines," provides fundamental construction flows and procedures for building Sabo facilities. This guideline are intended to ensure that construction work is conducted safely and efficiently. When developing construction plans, it is essential to thoroughly consider knowledge gained from past projects, as well as the specific characteristics of the construction area.

ＩＳＳＮ　１３４６-７３２８
国総研資料　第 904 号
令　和　４　年　５　月

国土技術政策総合研究所資料

TECHNICAL NOTE of
National Institute for Land and Infrastructure Management
No.904　　　　　　　　　　　　　May 2022

砂防基本計画策定指針（土石流・流木対策編）解説（英訳）

土砂災害研究部　砂防研究室

Technical Guideline for Establishing Sabo Master Plan against Debris Flow and Driftwood

Sabo Planning Division
Sabo Department

国土交通省　国土技術政策総合研究所

National Institute for Land and Infrastructure Management
Ministry of Land, Infrastructure, Transport and Tourism, Japan

国土技術政策総合研究所資料
第 904 号　　　2022 年 5 月

Technical Note of NILIM
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概　要

本資料は，国総研資料第904号「砂防基本計画策定指針(土石流・流木対策編)解説」を英訳したものである。

キーワード：土石流、流木、砂防基本計画

Synopsis

This note is the translation of the technical note of NILIM No.904 (Manual of Technical Guidelines for Establishing Sabo Master Plan against Debris Flow and Driftwood, originally written in Japanese in April, 2016).

Keywords: Debris flow, Driftwood, Sabo master plan

Disclaimer

This is a tentative translation of the original written in Japanese in April, 2016. However, some portions are modified or are not translated in case that they seem to be impossible to understand for those who don't know the context of the Japanese system of the technology standard or the domestic situations in Japan.

If what is translated in English is found to be different from what is written in Japanese version, the latter is always true.

This tentative translation could be subject to be changed without any prior notice.

Table of Contents

General Principles

1. Purpose of the Guideline

The Technical Guideline for Establishing Sabo Master Plans against Debris Flows and Driftwood (hereinafter, “Guideline”) presents basic concepts of debris flow and driftwood countermeasures to prevent sediment disasters caused by debris flows and driftwood. It also presents the minimum required items to be applied, which is based on the “*Technical Criteria for River Works*”. This Guideline aims to keep and promote the technical level related to debris flow and driftwood countermeasures.

2. Contents of the Guideline

The Guideline presents the criterion for technical matters related to countermeasure plan, a part of the Sabo masterplan against debris flow and driftwood. Section 1 of the Guideline explains the basic concept of Sabo planning for debris flow and driftwood countermeasures, Section 2 presents basic items in debris flow and driftwood countermeasure planning, and Section 3 describes the debris flow and driftwood control plan. Further, Section 4 discusses the spatial distribution plan of countermeasure structures against debris flow and driftwood and Section 5 defines the sediment and driftwood removal plan. Furthermore, technical matters related to the design of Sabo structures are presented in the "*Technical Guideline for Designing Sabo Facilities against Debris Flow and Driftwood* ".

Contents of the Guideline are to be revised periodically according to the improvement of technology.

3. Application of the Guideline

The Guideline shall be applied to prepare the Sabo master plan against debris flow and driftwood. However, it may not be exerted in case the application of the Guideline is not rational. Further, if a more appropriate method is available, such method can be adopted.

Section 1. Overview

The Sabo master plan against debris flow and driftwood is formulated to protect human lives, property, living environment, and natural environment from sediment disasters caused by debris flows and driftwood, and to contribute to land conservation.

The plan is developed by clarifying and surveying the conditions of the river, natural environment, and characteristics of the history, culture, and economy of the area to be protected.

Explanation

The Sabo master plan against debris flow and driftwood is enacted based on the Guideline. The Sabo master plan against debris flow and driftwood must be based on comprehensive evaluations of the social environment, natural environment, culture, history, and other regional and economic characteristics of the target basin. The evaluation shall include the past occurrence of debris flows and driftwood in the debris flow-prone rivers.

Further, the Guideline can be applied with necessary modifications to target basins where debris flow may occur even outside of debris flow-prone river. However, in such case, appropriateness of application shall be carefully examined considering similarity of the phenomena in debris flow-prone river.

Most debris flows reach a point with riverbed slope of 2° (approximately 1/30) or more. However, runout distances of debris flows shall be determined based on past disasters in the target basin, condition of sediment deposition in the river, and maximum grain size.

The Sabo master plan against debris flow and driftwood is conducted by referring to the flow chart in Figure 1.

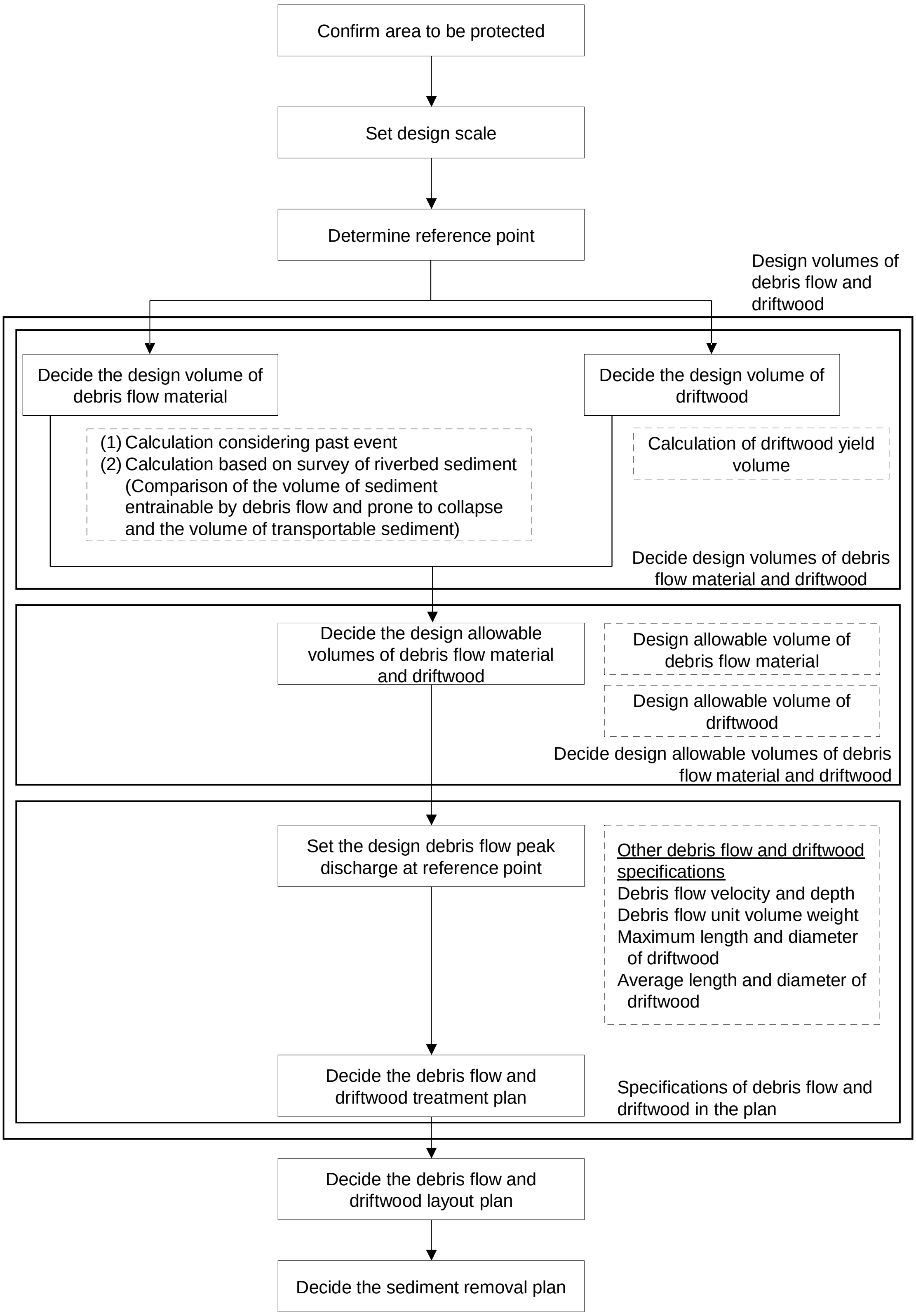

Figure 1 Flow chart of debris flow and driftwood countermeasure planning

Section 2. Basic items in debris flow and driftwood countermeasure planning

2.1 Basic guideline to planning

Debris flow and driftwood countermeasure planning shall be formulated to manage the debris flows and driftwood in a rational and effective manner. Thus, sediment disasters caused by debris flows and driftwood can be prevented.

Explanation

The purpose of countermeasures against debris flow and driftwood shall be achieved upon completion of the project based on the plan. Destructive force of debris flow and driftwood, and sediment inundation caused by blockage of narrow section and/or bridge due to driftwood has an enormous impact on human life, houses, and infrastructures.

Non-structural measures such as warning and evacuation systems shall be implemented to protect human life, houses, and infrastructures from debris flow and driftwood before the completion of the Sabo project. They are also needed for the debris flow of which scale exceeds the design scale. The debris flow with design scale is typically calculated considering annual exceedance probability of rainfall.

Design debris flow and driftwood volumes shall be reviewed and the debris flow and driftwood countermeasure plan shall be revised if the condition of target basin is drastically changed by natural phenomena such as large-scale landslide, occurrence of debris flow, earthquake, slope instability due to volcanic eruption, or by artificial causes such as development of the basin and change of vegetation.

2.2 Objects of protection

The objects of protection in debris flow-prone rivers are the population, homes, farmlands, and public facilities in the debris flow risk zone. These objects shall be determined by considering the direction and distance from the reference point, and elevation difference between riverbed and foundation of properties.

Explanation

The objects of protection are determined based on the *Debris Flow-prone River and Debris Flow Risk District Survey Methods (tentative)* [1)]. The Guideline is applied with necessary changes to plan for Sabo structures in a target basin where debris flow may occur even outside of debris flow-prone rivers.

2.3 Design scale

The design scale of debris flow and driftwood countermeasure plan shall be assessed based on the volume of debris flow material or the exceedance probability of rainfall according to the target basin characteristics.

The Guideline shall not be applied to the following cases:

- debris flow formed by sediment produced by the large-scale collapse of a mountain slope,
- breaching of a natural dam following landslide,
- volcanic mudflow triggered by snow melts,
- volcanic mudflow due to breaching of a crater lake.

Explanation

In principle, debris flow due to the rainfall of the design scale's exceedance probability (in principle, 24-hour rainfall or daily rainfall with 1 / 100 annual exceedance probability) is estimated empirically and theoretically.

In a debris flow and driftwood countermeasure plan, the volume of "debris flow with design scale" and driftwood can be determined based on past disaster records on the volume of debris flow material in the river.

2.4 Reference point

Reference point is a point where the volumes of sediment and driftwood are determined. The point is fundamentally set on the upstream of the area to be protected.

If necessary in enacting the debris flow and driftwood control plan, supplementary reference points are set at locations where debris flow and driftwood countermeasure facilities is to be built, or at locations where the pattern of sediment transport changes (Figure 2), or at river junctions with tributaries. Further, the source and scouring, transport, and deposition zones of a debris flow are determined according to the conditions of the river.

Explanation

The reference point is generally set at the outlet of a valley on the upstream of the area to be protected, or the downstream end of the transport zone of a debris flow. If countermeasure facilities will be built in the debris flow's deposition zone, the reference point shall be set downstream from the said countermeasure facility. Meanwhile, supplementary reference points, typically set upstream of the reference point, are fundamentally set at the locations where the countermeasure facilities will be built.

Locations where the pattern of sediment transport changes are shown in Figure 2.

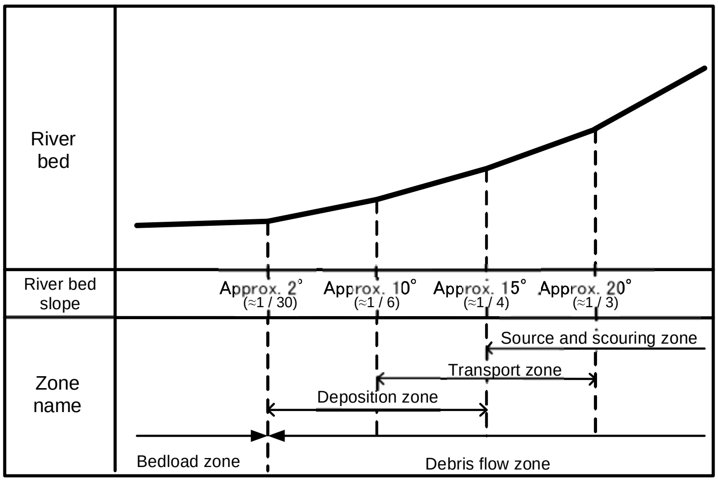

Figure 2 Changes of sediment transport pattern based on riverbed slope [1)]

2.5 Design volumes of sediment and driftwood

> The design volumes of sediment and driftwood include the design volume of debris flow material and driftwood, design allowable volumes of debris flow material and driftwood, and the peak discharge of debris flow.

Explanation

The design volumes of debris flow material and driftwood, design allowable volumes of debris flow material and driftwood, and the peak discharge of debris flow at the reference point need to be calculated to clarify the "debris flow with design scale" and driftwood.

The volumes of sediment and driftwood are calculated based on the Guideline. The method to calculate the volumes of sediment and driftwood at supplementary reference points and at locations where countermeasure facilities are located is explained in the Guideline, Section 2.6.

It should be noted that the effect of driftwood to peak discharge of debris flow, flow velocity, water depth, and unit weight is not considered.

2.5.1 Design volumes of debris flow material and driftwood

2.5.1.1 Design volume of debris flow material

The design volume of debris flow material is the volume of sediment transported by the "debris flow with design scale" at the reference point. The calculation shall be conducted by presuming that no debris flow and driftwood facilities exist upstream.

Explanation

The design volume of debris flow material is calculated based on the method in the Guideline, Section 2.6.1. L_{dy11} and L_{dy12} in Equations (2) and (4) represent the length of the river or channel from the reference point to their most upstream. The definition of a river and the method of determining the first-order stream are based on the *Debris Flow-prone River and Debris Flow Risk District Survey Method (Tentative)* [1)].

If the design volume of debris flow material calculated using the method in Section 2.6.1 is less than 1,000 m^3, the design volume of debris flow material shall be set to 1,000 m^3 [2)]. However, this approach shall not be applied to volume of debris flow material calculated at the supplementary reference points. Peak discharge of debris flow is calculated using the largest surge of debris flow. Calculation method of the peak discharge is explained in Section 2.6.3.

In a volcanic area, particularly if the volcano is active, the design volume of debris flow material must be revised according to the volcanic activity and the changes in the target basin situation.

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(Appendix A) Estimation method of the design volume of debris flow material in a small ephemeral river

In a small ephemeral river, volume of entrainable sediment by debris flow including the volume of sediment prone to collapse can be assessed accurately by a detailed survey of thickness of entrainable sediment using a (knocking) cone penetration test. Only in such case, the volume of sediment based on the survey shall not be round up to 1,000 m^3 even if the design volume of debris flow material is less than 1,000 m^3. Note that the term "small ephemeral river" refers to those that meet all the following conditions:

- An ephemeral river where the terrain has a shape with obscure or no lateral boundary to the flow path, water is not flowing constantly, and no sediment movement at normal times.
- The riverbed slope on the upper stream of the reference point is 10° or more, hence the whole basin is categorized as the transport zone and the source and scouring zone of debris flow (Referring to Figure 2).

〜〜

2.5.1.2 Design volume of driftwood

The design volume of driftwood is the volume of driftwood discharged to the reference point which is transported by the "debris flow with design scale". This volume is calculated by assuming no debris flow and driftwood facility exists upstream.

Explanation

The design driftwood discharge is calculated based on the method presented in the Guideline, Section 2.6.2. The values of L_{dy13} and B_d in Equation (7) of Section 2.6.2 are the same with the values from Section 2.5.1.1.

2.5.2 Design allowable volumes of debris flow material and driftwood

2.5.2.1 Design allowable volume of debris flow material

> The design allowable volume of debris flow material is the volume of sediment transporting through the reference point without causing any damage downstream.

Explanation

In principle, the design allowable volume of debris flow material is 0 (zero).

However, if sediment does not cause any damage downstream and it can be controlled by debris flow torrent training work, its volume may be considered as the design allowable volume of debris flow material.

2.5.2.2 Design allowable volume of driftwood

> The design allowable volume of driftwood is the volume of driftwood that does not cause disaster downstream of the reference point.

Explanation

In principle, the design allowable volume of driftwood is 0 (zero).

2.5.3 Peak discharge of debris flow at the reference point

> The peak discharge of debris flow is the maximum value of the discharge when "debris flow with design scale" passes the reference point. This peak discharge is calculated by assuming that no debris flow and driftwood facility exists upstream.

Explanation

The peak discharge of debris flow is calculated based on the method in this Guideline, Section 2.6.3.

2.6 Method to survey and calculate sediment and driftwood volumes

2.6.1 Method to calculate design volume of debris flow material

The design volume of debris flow material shall be comprehensively determined based on topographical maps and records of past disasters, after a field survey is conducted. In principle, the design volume of debris flow material shall be the smaller of two values: the volume of sediment entrainable channel deposits by debris flow and prone to collapse in the target basin and the volume of transportable sediment by "debris flow with design scale".

If basin-wide detailed surveys on collapsed slopes, sediment yield, and volume of debris flow material in past disasters have been conducted in the entire watershed (including the target basin), the design volume of debris flow material may be determined based on the survey results.

Explanation

Design volume of debris flow material is calculated based on the result of basin-wide survey of landslides and field surveys. If the directly measured value of volume of debris flow material is available, the design volume of debris flow material shall be calculated by considering the measured value.

(1) Volume of sediment entrainable by debris flow and prone to collapse in the target basin (V_{dy1})

$$V_{dy1} = V_{dy11} + V_{dy12} \quad \cdots\cdots (1)$$

$$V_{dy11} = A_{dy11} \times L_{dy11} \quad \cdots\cdots (2)$$

$$A_{dy11} = B_d \times D_e \quad \cdots\cdots (3)$$

Where,

V_{dy1} : volume of sediment entrainable by debris flow and prone to collapse in the target basin (m^3),

V_{dy11} : volume of entrainable channel deposits in the section from the reference point, supplementary reference point, or the point where the volume of debris flow material is to be calculated, to the furthest upstream point of the first-order stream (m^3),

V_{dy12} : volume of sediment prone to collapse (m^3),

A_{dy11} : average cross-sectional area of the entrainable channel deposits (m^2),

L_{dy11} : river length from the reference point, supplementary reference point, or the point where the volume of debris flow material is to be calculated, to the furthest upstream point of the first-order stream (m), as shown in Figure 3

B_d : average riverbed width where erosion is predicted to occur during debris flow (m),

D_e : average depth of the riverbed sediment where erosion is predicted to occur during debris flow (m).

B_d and D_e are estimated by referring to field survey and/or scouring situation during debris flow in nearby rivers. These values are used to calculate the sediment entrainable by debris flow and prone to collapse. To estimate B_d from the field survey (as shown in Figure 4(1)), it shall be estimated by classifying the riverbed deposition part and hillslope part. Classification of these parts can be referred to the change of river bank slope and difference of vegetation.

For estimation of D_e, not only cross-sectional situation of the sediment deposition but also the longitudinal continuity of the bedrock shall be considered by survey on bedrock exposure (Figure 4(1)). Figure 4(2) shows an example in a past debris flow disaster[3] as a reference for D_e estimation.

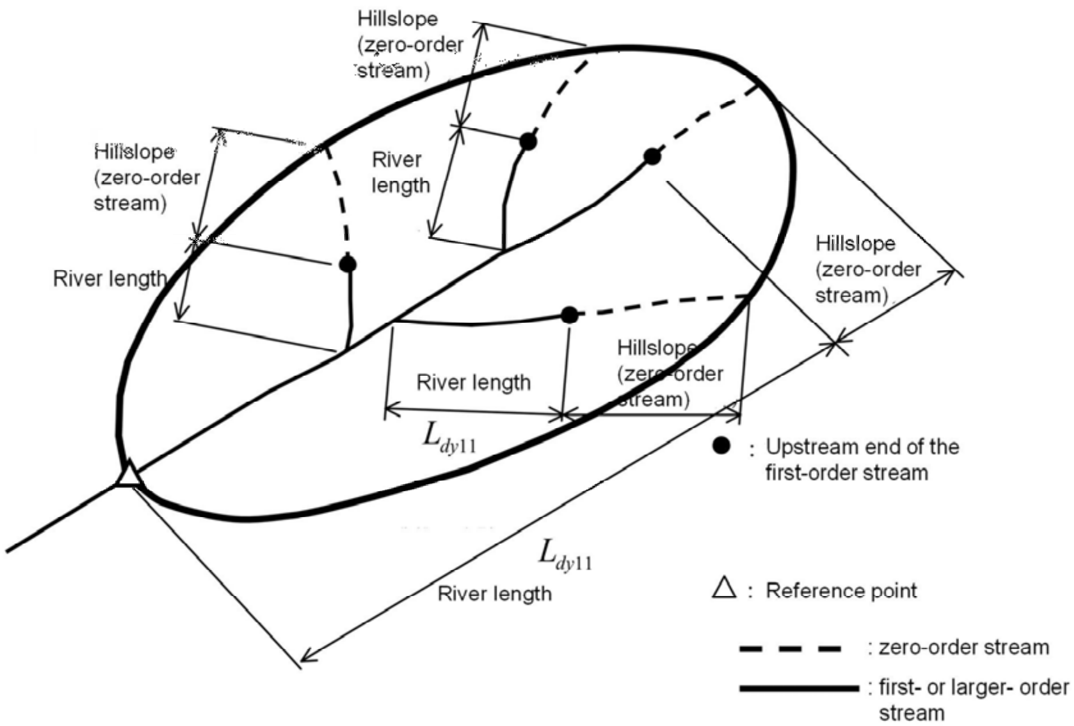

Figure 3 Measurement of L_{dy11}

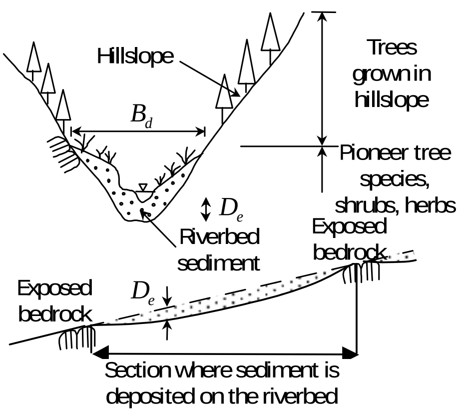

Figure 4(1) Method of measurement of erosion width and depth

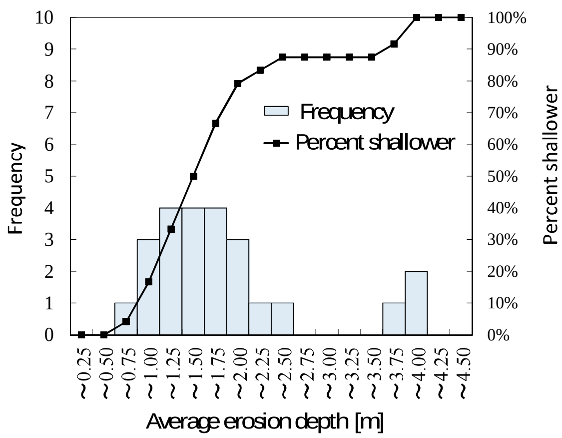

Figure 4 (2) Average erosion depth distribution

(Appendix B) Examples of average erosion depth (detailed example of Figure 4(2))

No.	Date	Prefecture	Cities	Name	Catchment Area (km^2)	Riverbed gradient (°)	Eroded Width (m)		Eroded Depth (m)		Maximum rainfall amount	
							mean	standard deviation	mean	standard deviation	1hour (mm)	24hour (mm)
1	Jul 29-30, 2011	Niigata	Minamiuonuma	Ubasawa	4.78	19.8	31.8	20.1	2.2	1.7	62.0	328.0
2	Jul 29-30, 2011	Niigata	Minamiuonuma	Futagosawa	0.78	27.0	27.6	13.0	3.9	2.4	62.0	328.0
3	Jul 29-30, 2011	Niigata	Minamiuonuma	Garasawa	1.60	22.4	10.0	5.9	1.1	0.7	62.0	328.0
4	Jul 29-30, 2011	Niigata	Minamiuonuma	Takatanasawa	0.82	23.6	15.9	7.0	3.7	2.2	58.3	321.2
5	Jul 29-30, 2011	Niigata	Minamiuonuma	Tsuchisawa	0.69	18.4	24.9	13.6	1.3	0.6	58.0	307.0
6	Sep 16-19, 2012	Mie	Inabe	Nishinogaitogawa	0.21	34.6	13.8	7.3	1.6	2.0	70.0	435.0
7	Sep 16-19, 2012	Mie	Inabe	Kotakigawa	1.39	25.3	22.6	5.8	3.9	2.0	70.0	435.0
8	Jul 11-12, 2012	Kumamoto	Aso	Daimonkawa	0.33	13.4	14.5	7.1	1.2	0.7	124.0	517.0
9	Jul 11-12, 2012	Kumamoto	Aso	Sakanashi area	0.09	19.3	42.2	19.3	1.6	1.3	124.0	517.0
10	Jul 11-12, 2012	Kumamoto	Aso	Shioigawa2	0.48	14.5	13.7	6.6	1.7	1.3	124.0	517.0
11	Jul 11-12, 2012	Kumamoto	Aso	Shinsyogawa3	0.07	28.2	16.9	6.9	1.0	0.6	83.0	417.0
12	Jul 11-12, 2012	Kumamoto	Aso	Doigawa	0.28	19.5	21.2	9.9	2.4	1.1	124.0	517.0
13	Jul 21, 2009	Yamaguchi	Hofu	Abetanikawa	0.53	15.0	16.0	5.7	1.9	0.9	60.0	266.0
14	Jul 21, 2009	Yamaguchi	Hofu	Yahatatanikeiryu	1.05	14.2	9.0	4.1	0.8	0.5	60.0	266.0
15	Jul 21, 2009	Yamaguchi	Hofu	Matugatanikawa	2.13	7.1	12.4	5.8	0.7	0.4	60.0	266.0
16	Jul 21, 2009	Yamaguchi	Hofu	Kamisatokawa	0.03	20.5	25.1	7.6	1.6	0.5	56.0	256.0
17	Jul 21, 2009	Yamaguchi	Hofu	Uedaminamikawa	1.10	12.2	15.9	8.0	1.1	0.6	60.0	266.0
18	Jul 8-11, 2014	Nagano	Nagiso	Nashisawa	2.27	18.4	25.6	11.6	1.8	1.2	76.0	143.0
19	Aug 9-10, 2013	Akita	Senboku	Kuyobutsusawa	0.03	17.0	41.7	10.3	1.3	0.9	58.0	189.0
20	Aug 19-20, 2014	Hiroshima	Hiroshima	Torigoekawa	0.34	16.2	15.9	7.1	1.0	0.5	87.0	247.0
21	Aug 19-20, 2014	Hiroshima	Hiroshima	Ueyamakawa	0.22	19.5	18.1	6.1	1.3	0.7	87.0	247.0
22	Aug 19-20, 2014	Hiroshima	Hiroshima	Yagibairinsawa	0.19	26.0	18.2	6.9	1.9	1.3	87.0	247.0
23	Aug 19-20, 2014	Hiroshima	Hiroshima	Jyourakuchikawa	0.03	22.3	18.9	5.4	1.3	0.5	87.0	247.0
24	Aug 19-20, 2014	Hiroshima	Hiroshima	Ⅰ-1-9-1010隣1	0.04	33.4	12.9	10.0	0.8	0.6	115.0	290.0

The volume of sediment prone to collapse (V_{dy12}) is calculated by one of the following methods.

(1-1) In case that the volume of sediment prone to collapse (V_{dy12}) can be estimated accurately

V_{dy12} in Equation (1) is the sediment volume (m^3) in the zero-order stream (ephemeral stream) and the hillslope which are predicted to collapse.

The zero-order stream is identified from contour lines using a 1/25,000 or larger-scale topographical map, or aerial LiDAR survey result. The zero-order stream is defined as a terrain where the depth (b) of the contour lines is less than its width (a), as shown in Figure 5.

In order to estimate the volume of sediment prone to collapse, the occurrence location, area, and collapse depth shall be estimated by considering the topographical and geological characteristics, distribution of the existing collapses, and results of field survey. There is a case [4] where site reconnaissance and (knocking) cone penetration test are carried out to estimate it. Moreover, boring survey will be useful.

Volume bulking of the collapsed soil due to destruction of skeleton is not considered.

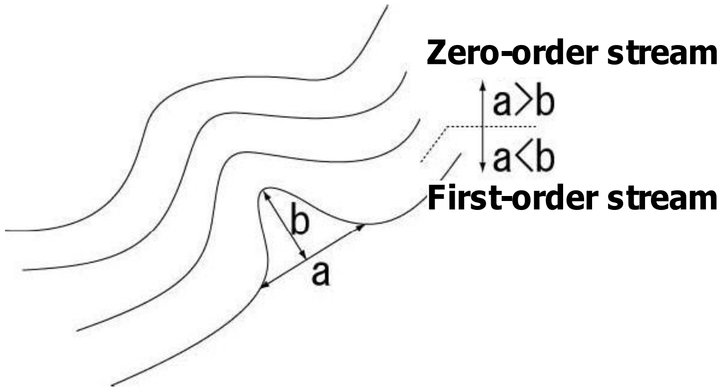

Figure 5 Topography of the zero-order stream

(1-2) In case that the volume of sediment prone to collapse (V_{dy12}) is difficult to be estimated accurately

The volume of sediment prone to collapse is estimated using the following equation considering the collapse in zero-order stream.

$$V_{dy12} \fallingdotseq \sum\left(A_{dy12} \times L_{dy12}\right) \quad (4)$$

$$A_{dy12} = B_d \times D_e \quad (5)$$

Where,

A_{dy12} : average cross-sectional area of the sediment prone to collapse in the zero-order stream (m^2),

L_{dy12} : river length from highest point of the first-order stream where the volume of debris flow material is calculated to the farthest point in the target basin (m) (Figure 6). If any tributary exists, its length shall be added.

If the sediment yield on hillslope and river banks is intense and the volume of sediment entrainable by debris flow and prone to collapse is expected to increase in the near future, the volume of this sediment yield needs to be estimated and cumulated, even though the volume of sediment prone to collapse is small, because the survey is conducted immediately after the occurrence of debris flows, for example.

(1-3) Survey to accumulate measured values

To calculate volume of transported sediment by considering past experiences, survey of flow condition when a debris flow occurs is necessary. For the field surveys on volume of transported sediment, aerial LiDAR or unmanned aircraft (e.g., drones) survey may be used. In particular, a method[6] to calculate volume of transported sediment by comparing aerial LiDAR survey data of before and after debris flow occurrence is usefull.

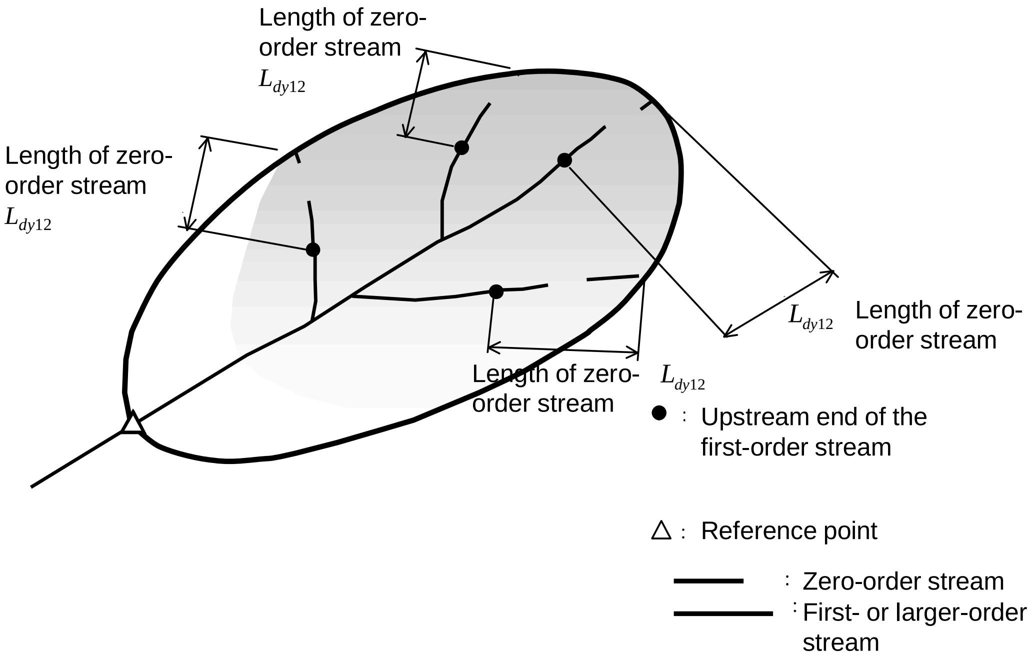

Figure 6 Measurement of L_{dy12}

(2) Volume of transportable sediment by the “debris flow with design scale” (V_{dy2})

Volume of transportable sediment by “debris flow with design scale” is calculated by obtaining the total water volume as the product of multiplying the rainfall of the annual exceedance probability (P_p, mm) by the target basin area (A, km^2). This total water volume is then multiplied by concentration of debris flow (C_d). Moreover, the discharge adjustment factor (K_{f2}) is also considered in the calculation.

$$V_{dy2} = \frac{10^3 \cdot P_p \cdot A}{1-K_v}\left(\frac{C_d}{1-C_d}\right)K_{f\,2} \quad \text{......(6)}$$

C_d is calculated with reference to the Guideline, Section 2.6.3. Although Equation (12) is Takahashi's Equation for a slope of 10-20°, it is assumed to be applied to gentle slope of lower than 10°. P_p is determined by studying the region’s rainfall characteristics and disaster characteristics. Typically, 24-hour rainfall is generally used. K_v is the porosity of about 0.4. K_{f2} is the discharge adjustment factor determined based on the target basin area (Figure 7), where the upper and lower limit is principally set to 0.5 and 0.1, respectively.

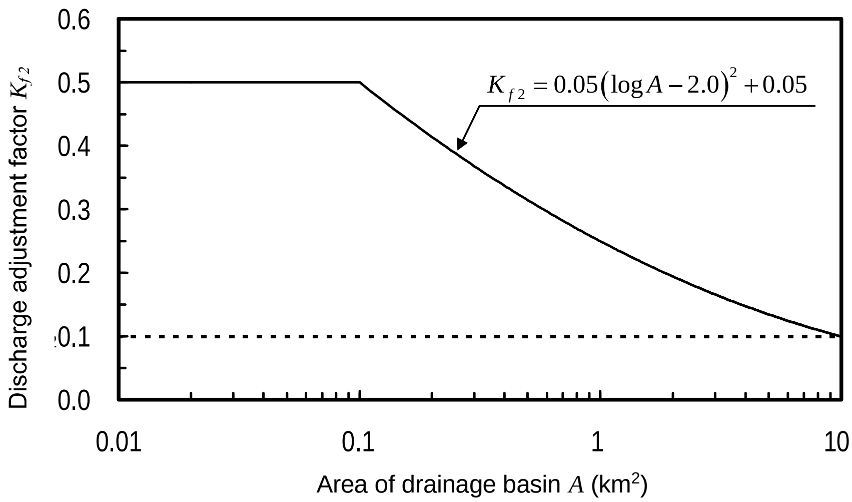

Figure 7 Discharge adjustment factor [4)]

2.6.2 Method to calculate design volume of driftwood

The design volume of driftwood is obtained by multiplying the estimated driftwood yield by the driftwood runoff ratio.

Explanation

The design driftwood runoff ratio (ratio of driftwood discharged to the exit of the valley to the driftwood yield) is reportedly between 0.8 and 0.9, when debris flow and driftwood facilities do not exist[7)]. The design volume of driftwood is the total tree volume calculated assuming no debris flow and driftwood facility exists in the target basin.

To clarify the design volume of driftwood, surveys of condition in the target basin, factors of yield, locations, quantity, length, and diameter of driftwood need to be conducted. Further, survey to estimate possible damages caused by driftwood also need to be carried out.

First, the surveys are begun with checking of the current condition of the target basin to clarify the condition of the forest.

Next, results of the survey of the forest condition are comprehensively assessed to hypothesize the factors causing driftwood yield.

Third, a survey to estimate the driftwood yield volume and location, and a survey to assess the volume, length, and diameter of driftwood that will flow down and deposit, are required to be conducted.

From these surveys, the damage caused by driftwood is estimated and the volume, length, and diameter of the driftwood that shall be controlled are determined.

(1) Survey of forest conditions

The standing trees, vegetation, and fallen trees (except log and lumber) are investigated preliminarily in the upstream area from the point where the design volume of driftwood is to be calculated.

(2) Survey of factors causing driftwood

The results of the survey of forest conditions are assessed comprehensively to determine the factors causing driftwood yield.

Determining the factors causing driftwood is vital to estimate the locations where driftwood will occur, its volume, length, weight, diameter, and the damage it will cause. If the slope is steep and brittle, debris flow or slope failure tends to occur during torrential rain. Along with such debris flow and slope failure, trees flow into river courses and become driftwood. Further, studying the damage caused by past cases of driftwood disaster is an effective method to estimate the factors causing driftwood.

Table 1 shows the factors causing driftwood yield.

Table 1 Causes of driftwood yield

Source of driftwood	Causes of driftwood yield
Discharge of standing trees	(1) Sliding of standing trees due to slope failure (2) Occurrence of debris flow and subsequent recruitment of standing trees (3) Erosion of river banks and/or riverbed by debris flow and recruitment of standing trees.
Discharge of fallen trees	(4) Recruitment of fallen trees due to pest damage or typhoon on river bed (5) Re-mobilization of fallen trees discharged in the past and remained in river bed (6) Re-mobilization of fallen trees discharged by snow avalanches

(3) Survey of the yield location, volume, length, and diameter of driftwood

The location of yield, volume, length, and diameter of driftwood are surveyed based on the causing factors from the field surveys of hillslope, interpretation of aerial photographs, and information of past disasters. Logs and timbers deposited on the riverbed are not included in the volume of driftwood yield.

(3-1) Causing factors and location of yield

A field survey is conducted, aerial photographs are interpreted, and past disasters are studied to determine the factors causing the yield and location where driftwood will occur.

(3-2) Calculation of driftwood yield volume based on field survey

Based on the determined driftwood causing factors and locations, the length and diameter of driftwood are surveyed to calculate the volume of driftwood yield.

The following method (herein this section called direct survey method) is used to directly survey the quantity, length, and diameter of trees and driftwood at the location where driftwood potentially occurs. In the direct survey method, it is necessary to estimate the location where driftwood will occur accompanied with debris flow and slope failure. For this estimation, the source and scouring zone and the transport zone where driftwood would occur need to be assessed based on Section 2.6.1 of this Guideline. After the source and scouring zone and the transport zone where driftwood would occur due to heavy rain is assumed, the quantity, length, and diameter of generated driftwood is estimated by surveying the standing trees, fallen trees, and driftwood quantity (number and volume) accumulated in the riverbed from the past disasters. Such survey is conducted by field survey and interpretation of aerial photographs, or their combination.

Direct survey method is classified into two sub-methods: (a) surveying all trees and driftwood within the range of the target area (hereinafter: complete survey method) and (b) surveying a number of samples at typical locations (hereinafter: sampling survey method). However, since complete survey method often requires hard effort to cover a wide target area, the sampling survey method is usually used.

Topographical maps and aerial photographs are used to interpret the approximate density, height, and species of trees in the source and scouring zone and transport zone of slope failure and/or debris flow. Based on this result, the source and scouring zone and transport zone are divided into several sub-zones so that each sub-zone has the same vegetation and forest type. Then, sampling survey (for an extent of 10 m × 10 m) is conducted in each sub-zone, where the number, species, height, and breast height diameter of the trees in each sub-zone are examined. The Guideline survey items are as follows:

(1) Density or number : number of trees, fallen trees and driftwood per 100 m^2
(2) Diameter : breast height diameter of trees, average diameter of fallen trees and driftwood.
(3) Length : height of trees, or lengths of fallen trees and driftwood

The volume of driftwood yield is calculated by the following procedure and equations. If the source and scouring zone and transport zone of debris flow consist of multiple forest type, the volume of driftwood yield (V_{wv}) is calculated for each forest type and then summed up. The width and length of the zero-order stream and the collapsed slope in the equation below conform to part 2.6.1 of the Guideline.

$$V_{wy} = \frac{B_d \times L_{dy13}}{100} \times \sum V_{wy2} \quad \text{......(7)}$$

$$V_{wy2} = \pi \cdot H_w \cdot R_w{}^2 \cdot \frac{K_d}{4} \quad \text{......(8)}$$

Where,

V_{wy} : volume of driftwood yield (m^3),
B_d : average width of riverbed where erosion is predicted to occur during a debris flow (m),
L_{dy13} : river length from the point where the driftwood yield is calculated to the farthest point in the target basin (m) as shown in Figure 8(1),
V_{wy2} : single tree volume (m^3),
ΣV_{wy2} : tree volume per 100 m^2 in the sampling survey (m^3/100 m^2),
H_w : tree height (m),
R_w : breast height diameter (m),
K_d : breast height coefficient (see Fig. 8 (2)).

If the aerial LiDAR measurement data of the target basin in the recent years is available, the height and number (density) of trees required to calculate the volume of driftwood can be extracted from it. For instance, the aerial LiDAR data was used to determine the forest type classification and volume of driftwood yield in a survey that covers a wide area[8)].

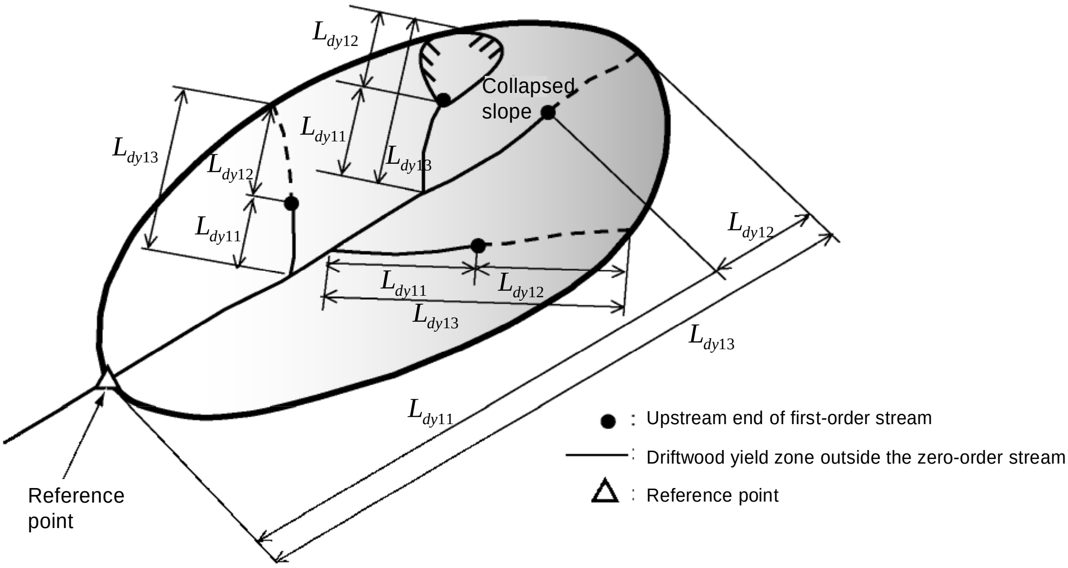

Figure 8(1) Length of driftwood yield zone (m） L_{dy13}

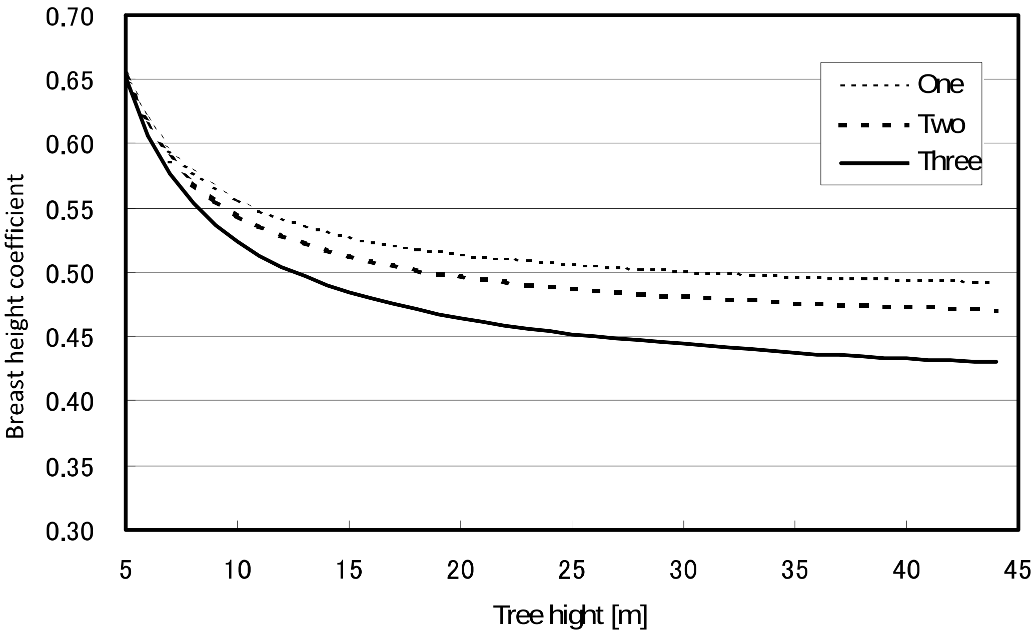

Figure 8(2) Breast height coefficient [9)]

(Remarks) One : Yezo spruce, Sakhalin fir

Two : Japanese cypress, sawara cypress, hiba arborvitae, umbrella pine

Three : Red cedar, pine, Japanese fir, Japanese hemlock, and other broad leafed and coniferous trees

Source: Plotted based on data in (Mine Ichizo (1958): *Forest Mensuration*, Asakura Publishing, page 146)

(3-3) Calculation of the volume of driftwood yield based on measured values

If there are cases of driftwood generated in neighboring rivers and related data about the volume of driftwood yield per unit of area of the target basin (V_{wy1}, m^3/km^2) is available, the design volume of driftwood can be calculated by the following equation.

$$V_{wy} = V_{wy1} \times A \quad \text{......(9)}$$

Where A: target basin area (km^2) (basin area with riverbed slope of 5° or more).

As a reference, Figure 9 shows the survey results of driftwood transported by debris flows. The figure shows the relationship between the target basin area of the river and the driftwood yield by coniferous and broad-leaved trees. If V_{wy1} is set as approximately 1,000 m^3/km^2, the plots obtained from past events could be roughly covered (Figure 9).

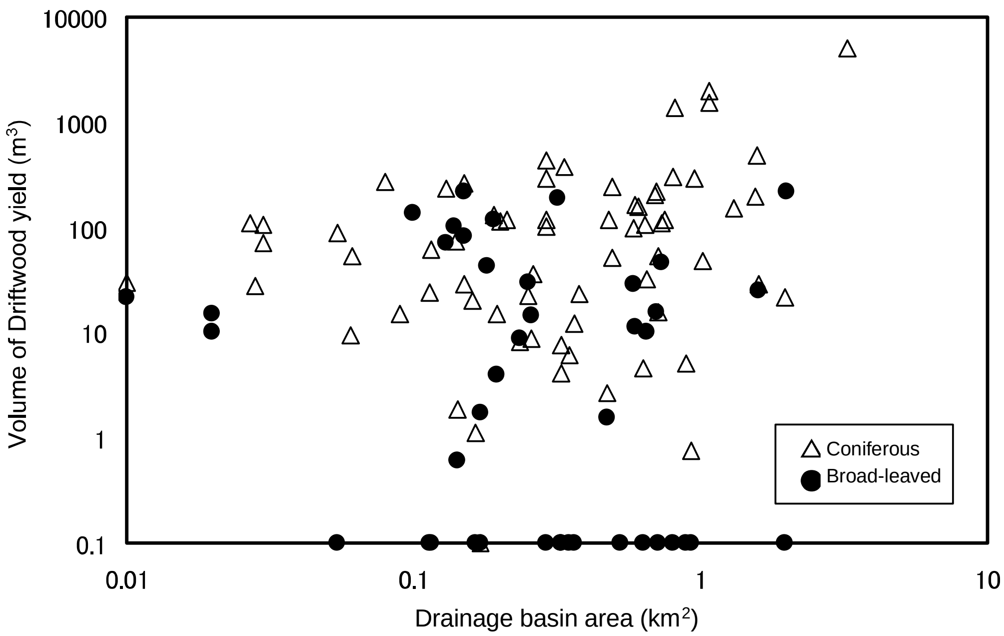

Figure 9 Target basin area –volume ofdDriftwood yield

2.6.3 Method to calculate peak discharge of debris flow

The peak discharge of debris flow can be obtained based on the volume of debris flow material. However, if the peak discharge of debris flow can be estimated using another method i.e., measured values for the same target basin is available, this value may be used.

Explanation

(1) Setting of the peak discharge of debris flow based on the volume of debris flow material

Figure 10 shows the relationship of total sediment volume containing in a debris flow with its peak discharge based on the observed peak discharge data of debris flows at Mt. Yakedake, Sakurajima, for example. The relationship of the average peak discharge with the total discharge of the debris flow is presented in Equation (10)[10)].

$$Q_{sp} = 0.01 \cdot \sum Q \quad \text{(10)}$$

$$\sum Q = \frac{C_* \cdot V_{dqp}}{C_d} \quad \text{(11)}$$

Where,

Q_{sp} : peak discharge of debris flow (m^3/s),

ΣQ : total debris flow volume containing in a debris flow (m^3),

V_{dqp} : sediment volume discharged by the largest surge of debris flow (including void) (m^3),

C_d : concentration of debris flow,

C_* : volumetric concentration of sediment deposited on the riverbed (approximately 0.6).

The lower limit of V_{dqp} is 1,000 m^3. This value applies to the design of all the debris flow and driftwood countermeasure facilities except when Section 2.5.1.1 of this Guideline "(Appendix A) Estimation method of the design volume of debris flow material on hillslopes" is applied.

The concentration of debris flow is obtained by the following equilibrium concentration equation[11)].

$$C_d = \frac{\rho \tan\theta}{(\sigma - \rho)(\tan\phi - \tan\theta)} \quad \text{(12)}$$

Where,

σ : density of gravel (approx. 2,600 kg/m^3),

ρ : density of water (approx. 1,200 kg/m^3),

ϕ : internal friction angle of sediment deposited on a riverbed (degrees) (approximately 30-40°, 35° is typically used),

θ : riverbed surface slope (degrees).

The riverbed surface slope used to calculate the debris flow peak discharge is the riverbed surface slope at the point where the "volume of sediment predicted to be transported by the largest surge of debris flow" is calculated. It should be no less than 10° which is considered to be the lower end of the transport zone. Note that the riverbed slope is basically the average riverbed slope in the section of about 200 m upstream of the planned location. This riverbed slope is calculated from the topography of before the facility being designed. If the section of about 200 m upstream of the planned location cannot represent the riverbed slope, a section shall be selected as the alternative according to the condition of the river.

When the calculated value (C_d) is larger than $0.9C_*$, C_d is assumed to be equal to $0.9C_*$. Meanwhile, if the calculated value (C_d) is smaller than 0.3, C_d value is assumed to be 0.3

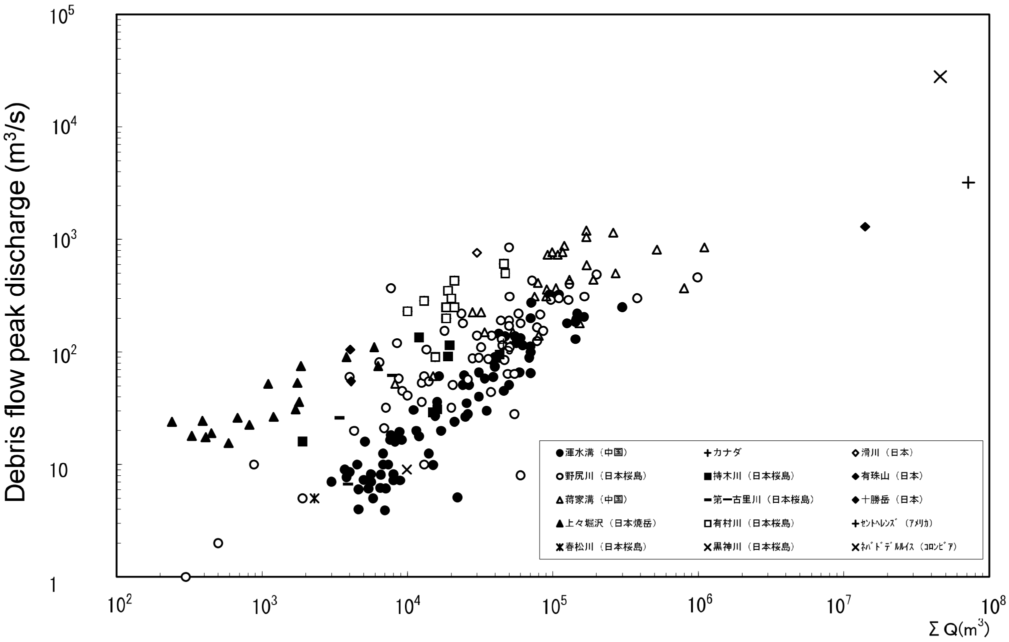

Figure 10 Correlation of peak discharge with ΣQ[10] (ΣQ is written as Q_T in the original work)

(2) Survey to collect measured values

To calculate the debris flow peak discharge using the measured values, survey of the actual condition of debris flow peak discharge is necessary. The following methods can be used to determine the peak discharge of debris flow by measured values.

1) Estimation from debris flow tracks

If traces and cross section of debris flow are apparent, flow velocity and the largest depth of debris flow are estimated and peak discharge is calculated.

2) Estimation from the velocity obtained from video image analysis

If a video record of the debris flow is available, it shall be analyzed to calculate the flow velocity. Field survey shall be conducted at the point where the flow velocity is calculated, and then the cross-section of flow is estimated. Peak discharge is calculated by multiplying the cross-sectional area of flow with the flow velocity. Another method is direct measurement of water level using a non-contact water level gauge to estimate the cross-section of flow.

* Calculation method of sediment volume discharged by the largest surge of debris flow V_{dqp}

Depending on surveys of past disasters, there are few cases where sediment is being discharged simultaneously from all tributaries. Hence, the maximum value of a peak discharge of debris flow is estimated from the maximum volume of sediment among multiple debris flow surges that occur during one flood event.

The sediment volume discharged by the largest surge of debris flow (V_{dqp}) is the smaller volume of the sediment entrainable by debris flow and prone to collapse and the transportable sediment by the largest surge of debris flow. For the estimation of V_{dqp}, it is assumed that there is no other Sabo facility in upstream of the point where the “sediment volume discharged by the largest surge of debris flow” is estimated. Sediment entrainable by debris flow and prone to collapse is calculated for each tributary in the upstream of the point where the “sediment volume discharged by the largest surge of debris flow” is calculated. Then, the largest value among these volumes is selected. Meanwhile, the transportable sediment by the design debris flow is calculated using Equation (6) but with the area (A) upstream of the point where the “sediment volume discharged by the largest surge of debris flow” is calculated.

Note that the point where V_{dqp} is calculated is different from that where the design volume of debris flow material is estimated.

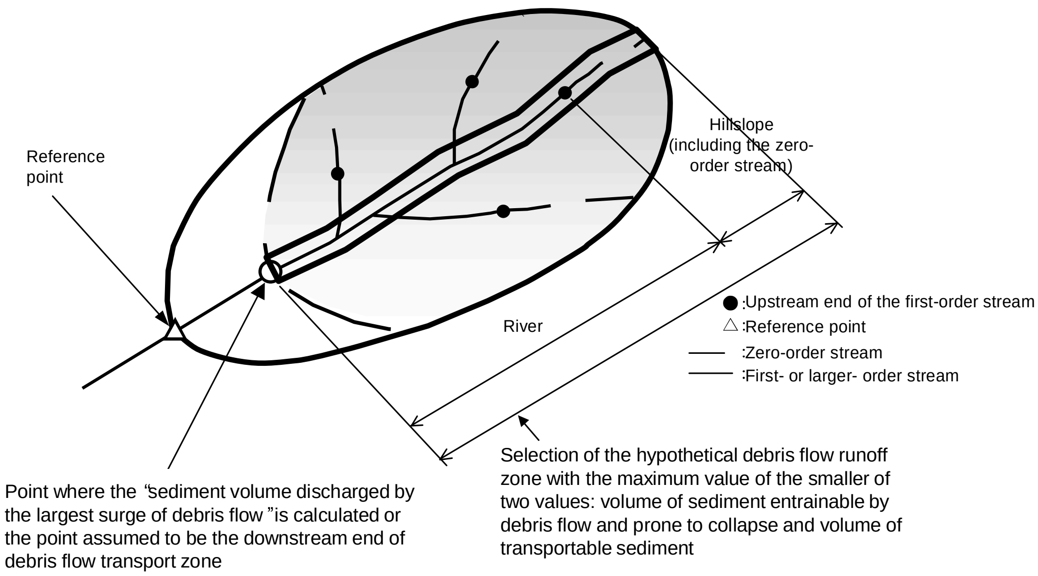

Figure 11 Mindset of hypothetical debris flow transport zone

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(Appendix C) Calculation of the peak discharge of debris flow based on rainfall

The debris flow mechanisms are assumed as follows: (1) riverbed deposits are severely eroded by flowing water which then become a debris flow, (2) collapsed hillslope sediment transformed to a debris flow as it is, and (3) the collapsed hillslope sediment blocks flowing water by forming a natural dam, creating a debris flow when the natural dam breaks. The rainfall-based calculation is a method to obtain the peak discharge of debris flow in mechanism (1) by predicting the peak discharge of water without sediment hydrologically, as explained below. Values of peak discharge calculated by Equation (10) and Equation (14) mentioned below vary depending on target basin area, rainfall, and volume of transported sediment.

If the ratio of the design volume of debris flow material to the area of target basin is 100,000 m^3/km^2, the 24-hour or daily rainfall (P_p) is 260 mm, and the target basin area is 1 km^2 or less, the result of the theoretical equation is smaller than the empirical equation.

The peak discharge of debris flow is obtained based on rainfall as follows.

$$Q_{sp} = K_q \cdot Q_p \quad \text{.......} (13)$$

Where,

Q_{sp} : peak discharge of debris flow (m^3/s),

Q_p : peak discharge of water without sediment under rainfall with the design exceedance probability (m^3/s),

K_q : coefficient.

The peak discharge of debris flow Q_{sp} (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Q_p (m^3/s) [12]:

$$Q_{sp} = \frac{C_*}{C_* - C_d} \cdot Q_p \quad \text{.......} (14)$$

(Example of peak discharge of debris flow calculation)

When σ = 2,600 (kg/m^3), ρ = 1,200 (kg/m^3), ϕ = 35°, and tan θ = 1/6, then the C_d value based on Equation (12) is 0.27. This value is smaller than 0.3, thus C_d value is assumed to be 0.3 and $Q_{sp} = 2 \times Q_p$ based on Equation (14).

〜〜

2.6.4 Method to calculate the peak discharge of water without sediment

The peak discharge of water without sediment shall be calculated by rational equation.

① Flood concentration time

The flood concentration time is, in principle, obtained by the following equation [13].

$$T_f = K_{p1} \cdot A^{0.22} \cdot P_e^{-0.35} \quad \text{.......(15)}$$

Where, T_f: flood concentration time (min), A: target basin area (km^2), P_e: effective rainfall intensity (mm/h), and K_{p1}: coefficient = 120.

② Average rainfall intensity

The average rainfall intensity during the flood concentration period is obtained from the 24-hour rainfall as shown by the following Mononobe equation[14].

$$P_a = \frac{P_{24}}{24}\left(\frac{T_f}{24}\right)^{K_{p2}} \quad \text{.......(16)}$$

Where, P_a: average rainfall intensity during the flood concentration time (mm/h), P_{24}: 24-hour rainfall (if P_{24} cannot be obtained, it can be substituted by daily rainfall (P_{day}) ($P_{24} \approx P_{day}$), K_{p2}: constant (K_{p2} = -½).

③ Effective rainfall intensity

Effective rainfall intensity is obtained by the following equation.

$$P_e = K_{f1} \cdot P_a \quad \text{.......(17-1)}$$

Where K_{f1}: peak discharge coefficient. If K_{p2} is assumed to be -½, the effective rainfall intensity is obtained by the following equation based on T_f and P_a.

$$P_e = \left(\frac{P_{24}}{24}\right)^{1.21} \cdot \left(\frac{24 \cdot K_{f1}{}^2}{\frac{K_{p1}}{60} \cdot A^{0.22}}\right)^{0.606} \quad \text{.......(17-2)}$$

④ Peak discharge of water without sediment

The peak discharge of water without sediment is obtained by Rational equation such as the following equation.

$$Q_p = \frac{1}{3.6} \cdot K_{f1} \cdot P_a \cdot A = \frac{1}{3.6} \cdot P_e \cdot A \quad \text{.......(18)}$$

2.6.5 Method to calculate debris flow velocity and depth

> The debris flow velocity and depth shall be estimated based on theoretical equation, empirical equation, and measured values.

Explanation

(1) Setting of debris flow velocity and depth based on debris flow peak discharge

The velocity of a debris flow U (m/s) can be represented by Manning's equation as shown below. This is a result obtained from the compiled observation results at Mt. Yakedake, Namekawa River, and Sakurajima Island[11].

$$U = \frac{1}{K_n} D_r^{2/3} \left(\sin\theta\right)^{1/2} \quad \text{......(19)}$$

Where,

D_r : debris flow hydraulic radius (m),

θ : riverbed surface slope (degrees),

K_n : roughness coefficient (s.m$^{-1/3}$).

The riverbed surface slope (θ) is adequately determined based on Table 2 depending on the items for usage. The value of the roughness coefficient (K_n) is much larger than that for the flow of the water without sediment. The K_n for the front part of debris flow flowing through a natural river course is 0.10 [15]. The velocity (U) and depth of the debris flow (D_d, here D_d = D_r for convenience) are obtained for the front part.

The debris flow depth D_d (m) is obtained by simultaneously solving Equations (19), (20), and (21) based on the width of the flow B_{da} (m) and the peak discharge of debris flow Q_{sp} (m^3/s).

$$Q_{sp} = U \cdot A_d \quad \text{......(20)}$$

Where, A_d: cross-sectional area of debris flow peak discharge (m^2).

In general, the debris flow due to the rainfall of the exceedance probability of the design scale flows through entire cross-section of the river. Therefore, the width of the debris flow B_{da} is shown in Figure 12, while the depth of the debris flow D_d is obtained by Equation (21).

$$D_d = \frac{A_d}{B_{da}} \quad \text{......(21)}$$

The velocity and depth of debris flow is calculated by using the average profile of three to five cross sections. These cross sections are randomly selected between the dam location and the upstream end of sedimentation, or the downstream end of the debris flow source and scouring zone. The average profile is often obtained by approximating each profile to trapezoids. However, if the profiles of cross sections quite differ each other and taking average might lead underestimation of debris flow momentum, the profile of cross section to calculate A_d shall be selected carefully. Moreover, if the profiles of cross section in the section where debris flow will deposit differs from that in the section upstream, the same is true.

(2) Survey for collecting measured values

The following methods are used to obtain the actual velocity of debris flow.

1) Debris flow velocity calculation from video or other image analyses

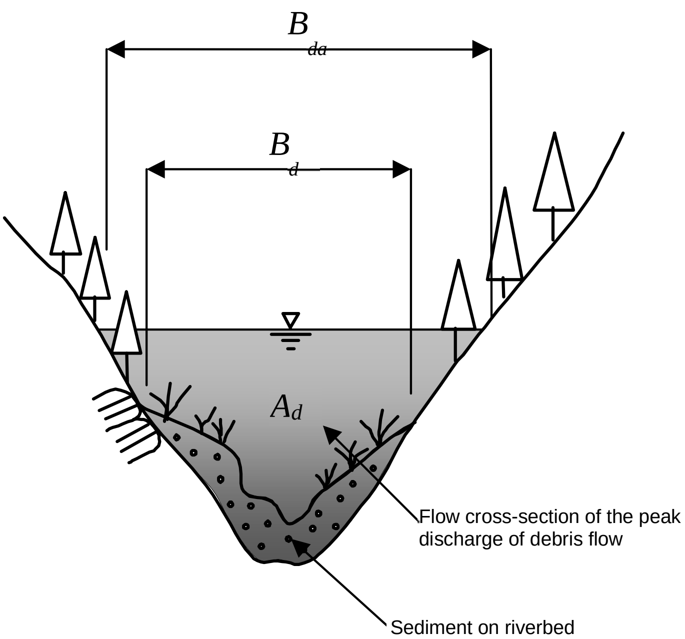
If a video that records the flow condition of debris flow is available, the flow velocity can be calculated by analyzing it.

2) Method of estimation from the tracks of debris flow on a river bend

If the debris flow drifts on outer bank on a river bend and the drift height can be surveyed on-site, the velocity of debris flow can be estimated based on the method of designing the curved section of debris flow torrent training work[16, 17].

Table 2 Definition of riverbed surface slope (θ)

Items for usage	Riverbed surface slope
When calculating the design external forces to perform stability analysis and structural calculations of the main structure and its wings: Concentration of debris flow (C_d) Debris flow velocity (U) Debris flow depth (D_d)	Riverbed surface slope before the facilities being built (θ_o)
When setting the spillway of a Sabo dam that allows the peak discharge of debris flow to pass: Overflow depth (D_d)	Design deposit surface slope (θ_p)



※ B_d: average river width where erosion is predicted to occur during a debris flow (See Fig. 4(1))

Figure 12 Image of debris flow cross-section and width of the flow B_{da}

2.6.6 Method to calculate debris flow unit weight

The debris flow unit weight shall be estimated based on the measured values and on empirical and theoretical research.

Explanation

The debris flow unit weight γ_d (kN/m^3) is obtained by the following equation.

$$\gamma_d = \{\sigma \cdot C_d + \rho \cdot (1 - C_d)\} g \quad \text{......(22)}$$

Where g: gravity acceleration (9.81 m/s^2), and γ_d: debris flow unit weight (kN/m^3). C_d is obtained by Equation (12).

(Note) Example of the debris flow unit weight measurement

To obtain the debris flow unit weight, several methods[18)] such as water gauge and load cell are available. Observation data from these methods have been accumulated.

2.6.7 Method to calculate drag force of debris flow

Drag force of debris flow is estimated using the flow velocity, depth, and unit weight of a debris flow.

Explanation

The drag force of debris flow is obtained by the following equation.

$$F = K_h \cdot \frac{\gamma_d}{g} \cdot D_d \cdot U^2 \quad \text{.........................} (23)$$

Where,

F : drag force of debris flow per unit width (kN/m),

U : flow velocity of debris flow (m/s),

D_d : depth of the debris flow (m) obtained according to the Guideline, Section 2.6.5,

g : gravity acceleration (9.81 m/s^2),

K_h : coefficient (= 1.0), and

γ_d : unit weight of the debris flow (kN/m^3).

2.6.8 Method to calculate maximum boulder size

The maximum boulder size is estimated from field survey results.

Explanation

The maximum boulder size is used to design the overflow section and permeable section, and to obtain the impact load of boulder for structure design of a dam according to the *Technical Guideline for Designing Sabo Facilities against Debris Flow and Driftwood* .

To calculate the maximum boulder size, a frequency distribution is created by measuring the diameters of a total of more than 200 boulders located in the sections of 200 m upstream and 200 m downstream from the planned position of the Sabo dam. The 95% maximum size (D_{95}) in the distribution is defined as the maximum boulder size. Boulders shall be measured from the boulder groups on the riverbed which is considered to be the deposit of the front part of debris flow. It is noted that the obtained maximum boulder size (D_{95}) should be checked whether it is the representative value of the grain size distribution of the boulders around the location of the designed facility. If less than 200 boulders exist, the target grain size range of boulders (>256 mm) in the survey area shall be expanded until it reaches to 200 boulders, such as including smaller gravels Boulders that are angular, comprised of different material, considered to have certainly rolled down from the hillslope, and predicted not to mobilize as debris flow are excluded from measurement.

2.6.9 Method to calculate maximum length and diameter of driftwood

The maximum length and diameter of driftwood shall be estimated based on the survey result of driftwood discharge volume. Estimation of maximum length of driftwood shall consider the average flow width of the debris flow.

Explanation

The maximum length and diameter of driftwood are used to calculate the impact of driftwood in examining the structure of debris flow and driftwood facilities in the *Technical Guideline for Designing Sabo Facilities against Debris Flow and Driftwood* . The maximum length of driftwood is used to determine the spacing of the members in the driftwood retention work. The maximum length of driftwood, L_{wm} (m), is estimated by the following equations, assuming that the average width of debris flow is the “average riverbed width where erosion is predicted to occur during a debris flow”, B_d (m), and that the maximum tree height of standing trees runoff from upstream is H_{wm} (m).

In case $H_{wm} \geq 1.3B_d$, then $L_{wm} \approx 1.3B_d$

In case $H_{wm} < 1.3B_d$, then $L_{wm} \approx B_d$

The maximum diameter of driftwood, R_{wm} (m), is assumed to be almost identical to the maximum breast height diameter of standing trees which is predicted to become driftwood in the upstream area (breast height diameter of the 95% largest tree of the expected driftwoods). In addition, fallen trees (except logs and timbers) that are expected to become driftwood shall also be surveyed. Underestimation of the maximum diameter shall be avoided.

2.6.10 Method to calculate average length and average diameter of driftwood

The average length and average diameter of driftwood are estimated based on the survey results of design volume of driftwood (2.6.2 of the Guideline). The minimum flow width of the debris flow shall be considered when estimating the average length of driftwood.

Explanation

The average length of driftwood, L_{wa} (m), is calculated by the following equations, assuming that the minimum flow width of the debris flow is B_{dm} (m), and the average tree height of standing trees discharged from upstream is h_{wa} (m).

In case $h_{wa} \geq B_{dm}$, then $L_{wa} \approx B_{dm}$

In case $h_{wa} < B_{dm}$, then $L_{wa} \approx h_{wa}$

The average diameter of driftwood, R_{wa} (m) is assumed to be identical with the average breast height diameter of standing trees in the upstream area that are predicted to become driftwood.

Section 3. Debris flow and driftwood control plan

A debris flow and driftwood control plan is enacted for each debris flow-prone river so that the "debris flow with design scale" and contained driftwood can be rationally and effectively controlled.

Explanation

Debris flow and driftwood control plan is a plan to treat the design volumes of sediment and driftwood of Sabo facilities (hereinafter the "debris flow and driftwood countermeasure facilities"). These design volumes include the design capturing volume of debris flow and driftwood (design sediment capturing volume, design driftwood retention volume), design depositing volume of debris flow and driftwood (design sediment depositing volume, design driftwood depositing volume), and design volume of debris flow and driftwood prevention (design sediment prevention volume, design driftwood prevention volume).

3.1 Basics of enacting debris flow and driftwood control plan

A debris flow and driftwood control plan shall be enacted by considering the design volumes of sediment and driftwood, pattern of sediment movement, topography, objects of protection, etc. In this plan, the spatial distribution of debris flow and driftwood countermeasures facilities is set so that debris flow and driftwood are treated rationally and effectively.

If a debris flow torrent training work is planned and the volume controlled by it is set as the design allowable volume of debris flow material (the Guideline, Section 2.5.2.1), the grain size of transported sediment shall be scrutinized before the work is adopted. This is to avoid deposition in the work and subsequent flooding.

Explanation

A debris flow and driftwood control plan, referring to Section 4.3.1.1, encompasses the following volumes:

・design volume of debris flow material and driftwood (*V*),
・design allowable volume of debris flow material and driftwood (*W*),
・design capturing volume of debris flow material and driftwood by the debris flow and driftwood countermeasure facilities (*X*),
・design depositing volume of debris flow material and driftwood (*Y*), and
・design volume of debris flow and driftwood prevention (*Z*).

This debris flow and driftwood control plan shall satisfy Equation (24).

$$V - W - \left(X + Y + Z\right) = 0 \quad \text{.....................(24)}$$

Where *V*, *W*, *X*, *Y*, and *Z* are calculated by the following equations.

$$V = V_d + V_w \quad \text{...(25)}$$
$$W = W_d + W_w \quad \text{...(26)}$$
$$X = X_d + X_w \quad \text{...(27)}$$
$$Y = Y_d + Y_w \quad \text{...(28)}$$
$$Z = Z_d + Z_w \quad \text{..(29)}$$

Where,

V_d : design volume of debris flow material (m^3),
V_w : design volume of driftwood (m^3),
W_d : design allowable volume of debris flow material (m^3),
W_w : design allowable volume of driftwood (m^3),
X_d : design sediment capturing volume (m^3),
X_w : design driftwood retention volume (m^3),
Y_d : design sediment depositing volume (m^3),
Y_w : design driftwood depositing volume (m^3),
Z_d : design sediment prevention volume (m^3), and
Z_w : design driftwood prevention volume (m^3).

3.2 Design capturing volume of debris flow and driftwood

The design capturing volume of debris flow and driftwood is the volume of "debris flow with design scale" and contained driftwood captured by the debris flow and driftwood countermeasure facilities. The design capturing volume of debris flow and driftwood is the total of design sediment capturing volume and design driftwood retention volume.

Explanation

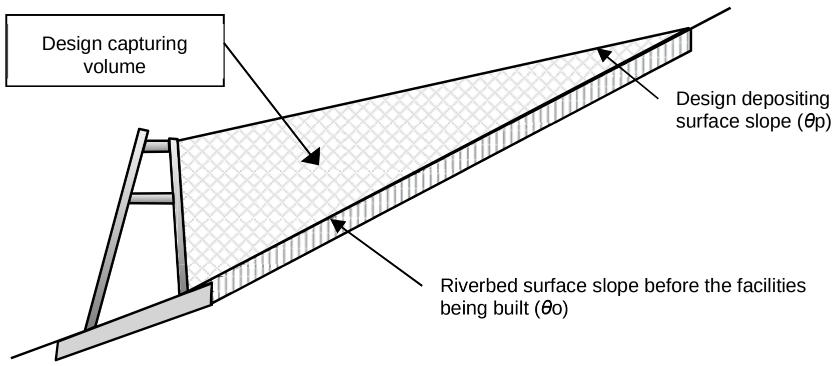
For open type Sabo dam, the design capturing volume of debris flow and driftwood is the space enclosed by the surface of the riverbed before the facilities being built and the assumed surface of the design depositing sediment (space indicated by diamond patterns in Figure 13). For closed type or semi-open type Sabo dam, the design capturing volume of debris flow and driftwood is the space enclosed by the surface of the depositing sediment in normal times and the assumed surface of the design depositing sediment (space indicated by diamond patterns in Figure 13).

The design depositing surface slope is generally 1/2 to 2/3 times of the riverbed surface slope before the facilities being built at the location of countermeasure facilities, depending on past records. However, since the "debris flow with design scale" is assumed not depositing on a slope steeper than 1/6, the maximum design depositing surface slope is 1/6 (tan θ). The deposit surface slope in normal times has an upper limit of half the riverbed slope before the facilities being built, according to past records.

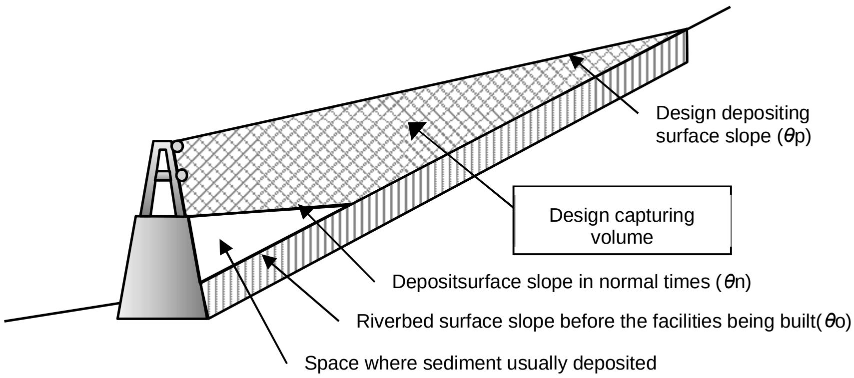
It is well known that the design deposit surface slope and deposit surface slope in normal times would be gentle in the region where specific geological condition exists such as decomposed granite and *shirasu*. Therefore, these design slopes shall be determined according to past records. Sediment that temporarily deposited on a steep slope used not to be re-eroded even in a long period, depending on the condition of water flow after the deposition. Therefore, design capturing volume of debris flow and driftwood shown in Figure 13 must be kept empty by sediment and driftwood removal works. The concept of the removal works is explained in Section 5 of the Guideline.

The concept of design capturing volume of debris flow and driftwood is shown in Figure 13.

• Open type Sabo dam



• Semi-open type Sabo dam



• Closed type Sabo dam

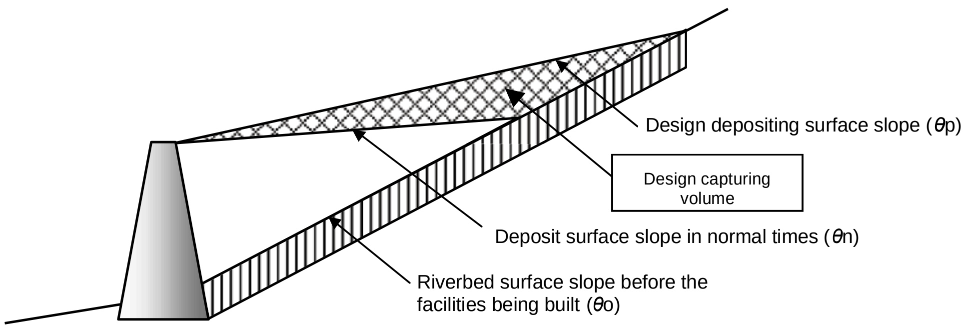


Figure 13 Concept of design capturing volume of debris flow and driftwood

3.2.1 Design sediment capturing volume

The design sediment capturing volume is the volume of sediment to be captured by debris flow and driftwood facilities in the volume of “debris flow with design scale” and contained driftwood.

Explanation

For open type Sabo dams, the design sediment capturing volume is equivalent to the space surrounded by the surface of riverbed before the facilities being built and the assumed surface of the design depositing sediment. While for closed and semi-open type Sabo dams, this space is surrounded by the surface of deposit in normal times and the assumed surface of the design depositing sediment. These spaces shall be kept empty by sediment and driftwood removal (diamond hatch pattern in Figure 13).

3.2.2 Design driftwood retention volume

The design driftwood retention volume is the volume of driftwood to be captured by debris flow and driftwood facilities in the volume of "debris flow with design scale" and contained driftwood.

Explanation

(1) Design driftwood retention volume of open type or semi-open type Sabo dam

Design driftwood retention volume for open type or semi-open type Sabo dam is calculated using Equation (30).

$$X_{w1} = K_{w11} \times X \text{ ......} \quad (30)$$

Where, X: design capturing volume of debris flow and driftwood by debris flow and driftwood facilities (m^3), X_{w1}: design driftwood retention volume by a main dam (m^3), K_{w11}: ratio of driftwood content to the design capturing volume of debris flow and driftwood (ratio of the design driftwood retention volume to the design capturing volume of debris flow and driftwood).

K_{w11} for open type or semi-open type Sabo dam is equal to the ratio of driftwood content (K_{w0}) to the design volume of debris flow and driftwood that is expected to flow into the dam (for K_{w0}, refer to this section (2)). Because open type or semi-open type Sabo dam captures both sediment and driftwood in debris flow at the same time, not selectively.

If the height of the opening part of a semi-open type Sabo dam is low, or the balance of the design driftwood retention volume in the opening part to the design driftwood depositing volume in the closed part under the opening part of the semi-open type Sabo dam is too small in other words, driftwood may not be sufficiently captured in the opening part due to ponding in the closed part. So that, in case that the total of the design driftwood retention volume in the opening part and the design driftwood depositing volume in the closed part exceeds the design capturing volume of debris flow and driftwood, the design driftwood retention volume and the design driftwood depositing volume of the semi-opening type Sabo dam shall be set to a value less than or equal to the driftwood retention volume in the opening part.

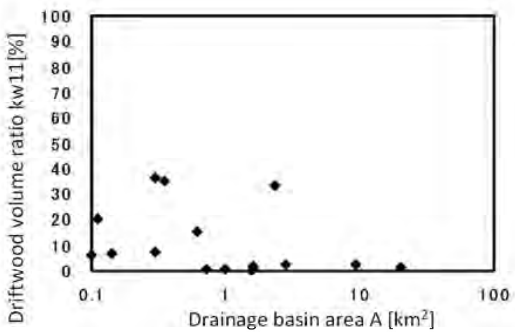


Figure 14 Driftwood volume ratio for open type Sabo dam

(2) Design driftwood retention volume by closed type Sabo dam

The design driftwood retention volume for the closed type Sabo dam is the smaller value between the result of Equation (31-1) and Equation (31-2). The design driftwood retention volume calculated by Equation (31-1) is obtained from the ratio of the design volume of driftwood to the design volume of debris flow material and driftwood which is expected to flow into the planned site for the Sabo dam. While the design driftwood retention volume calculated by the Equation (31-2) is obtained from the ratio of design driftwood retention volume to the design capturing volume of debris flow and driftwood in the Sabo dam.

Design driftwood retention volume by closed type Sabo dam

$$X_{w1} = K_{w0} \times X \times (1 - \alpha) \text{………(31-1)}$$

$$X_{w1} = K_{w11} \times X \text{………(31-2)}$$

Where,

X : design capturing volume of debris flow and driftwood by debris flow and driftwood facilities (m^3),

X_{w1} : design driftwood retention volume by the Sabo dam (m^3),

K_{w0} : ratio of driftwood content to the design volume of debris flow material and driftwood which is expected to flow into the dam,

α : ratio of driftwood volume overflowed Sabo dam (approx. 0.5),

K_{w11} : ratio of driftwood content to the design capturing volume of debris flow and driftwood (if there have not been capturing cases in the target river, K_{w11} ＝ 2% may be applied).

If a part of debris flow and driftwood is expected to be captured or retained by debris flow and driftwood countermeasure facilities upstream, K_{w0} shall be estimated after deduction of such volumes.

In experiments under certain conditions[19)], about a half of the driftwood that concentrated at the front part of the debris flow tends to overflow the closed type Sabo dam. Note that more data on the actual condition of driftwood discharge needs to be investigated to clarify its mechanism because driftwood transportation is a complicated phenomenon related to many factors including the flow condition of debris flow, riverbed slope around the Sabo dam, and the shape of depositing area.

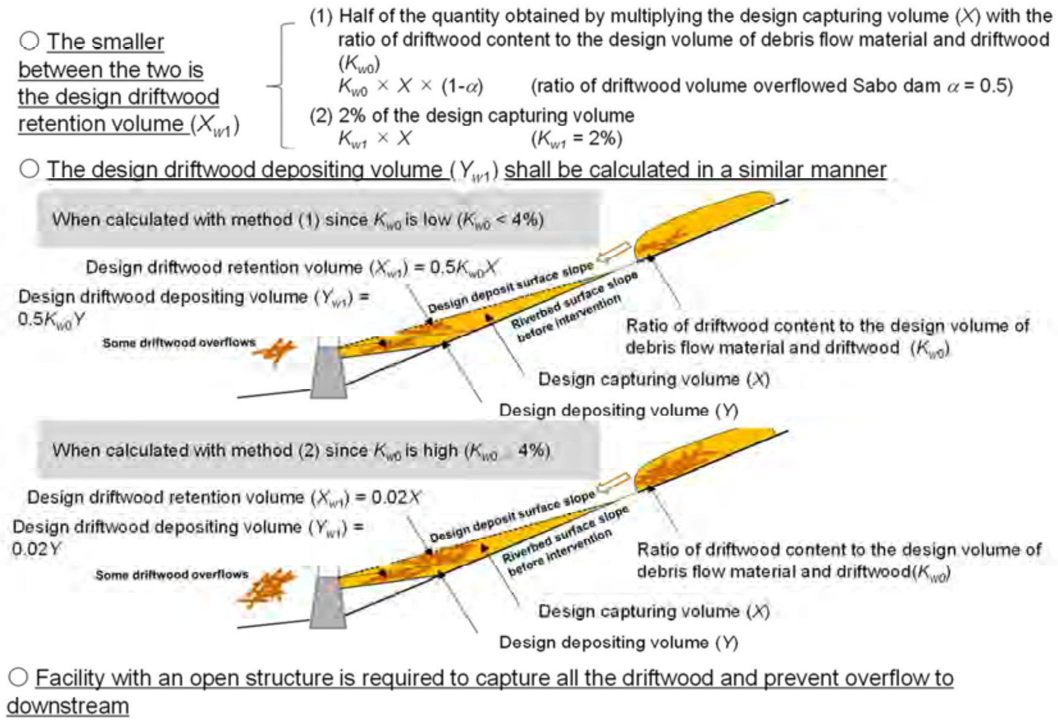


Figure 15 Outline of the design driftwood retention/depositing volume in a closed type Sabo dam

(In the case of only one Sabo dam is planned in a target basin)

In principle, driftwood shall be retained by the main dam of closed-type Sabo dam. However, if a driftwood capturing work is required at the sub dam due to the limitation of topographic conditions, the design driftwood retention volume is calculated with Equation (32).

The design driftwood retention volume (X_{w2}, unit: m^3) by the sub-dam (limited to cases where a driftwood capturing work is installed at the sub-dam).

$$X_{w2} = A_w \times R_{wa} \quad \text{......(32) (see the appendix D)}$$

Therefore,

$$X_w = X_{w1} + X_{w2} \quad \text{......(33)}$$

〜〜〜

(Appendix D) Design driftwood retention volume in bedload zone

In the case of driftwood retention works installed in bedload zone, the retention driftwood volume shall be calculated assuming that the driftwood covers the reservoir surface with the depth equivalent to the average diameter of driftwood, for the sake of brevity (actually the accumulation situation of driftwood would be various). The projected area of a retained driftwood on the horizontal plane is calculated based on the total of the average length of driftwood (L_{wa}) multiplied by the average diameter of driftwood (R_{wa}).

Further, the area (A_w) of the driftwood deposition or retention pond of the driftwood retention work which is required to retain driftwoods with the design driftwood retention volume is estimated by the following equation.

$$A_w \geq \Sigma(L_{wa} \times R_{wa}) \quad \text{......(34)}$$

The actual volume of the driftwood deposited on the depositing area or in the reservoir (V_{wc}) is represented by the following equation. However, the word “actual” in V_{wc} refers only to the volume of the driftwood, without any voids.

$$V_{wc} \approx A_w \times R_{wa} \quad \text{......(35)}$$

In the bedload zone, driftwood is considered to flow on the surface of flowing water, separated from the sediment. Thus, a closed type Sabo dam is assumed not to retain driftwood.

〜〜〜

3.3 Design depositing volume of debris flow and driftwood

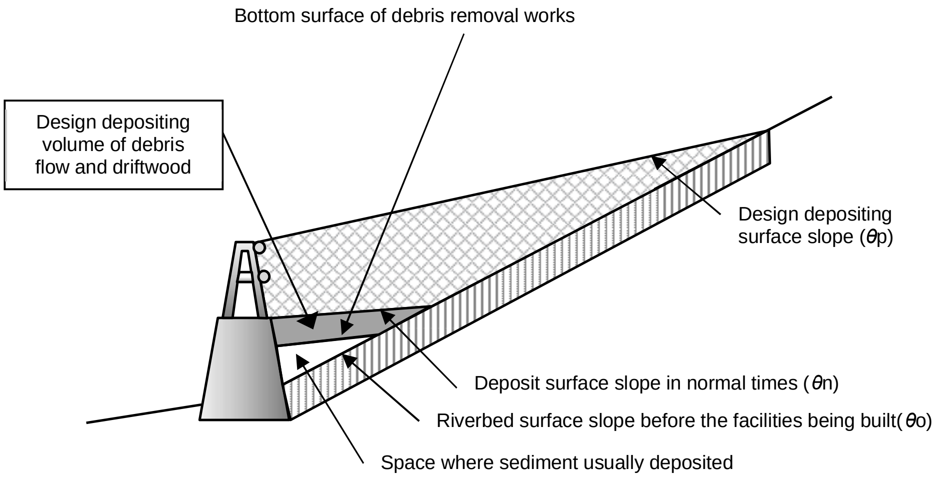
The design depositing volume of debris flow and driftwood is the volume of "debris flow with design scale" and contained driftwood that is deposited upstream the debris flow and driftwood facility. The design depositing volume is the total of design sediment depositing volume and design driftwood depositing volume. The space equivalent to the design depositing volume of debris flow and driftwood shall be kept empty by sediment and driftwood removal in accordance with the sediment and driftwood removal plan.

Explanation

The design depositing volume of debris flow and driftwood differs depending on the type of debris flow and driftwood facility. In closed type and semi-open type Sabo dams, it shall be deposited in the space surrounded by the riverbed surface before the facilities being built and the deposit surface in normal times (grey colored in Figure 16). For debris flow depositing work, see the Guideline, Section 4.3.4. In order to make sediment and driftwood deposit effectively, the space equivalent to the design depositing volume of debris flow and driftwood shall be kept empty by the sediment and driftwood removal work because deposition may be proceeded by normal runoff events. The concept of sediment and driftwood removal are explained in Section 5 of the Guideline.

The concept of the design depositing volume for closed type and semi-open type Sabo dam is presented in Figure 16.

- Semi-open type Sabo dam



- Closed type Sabo dam

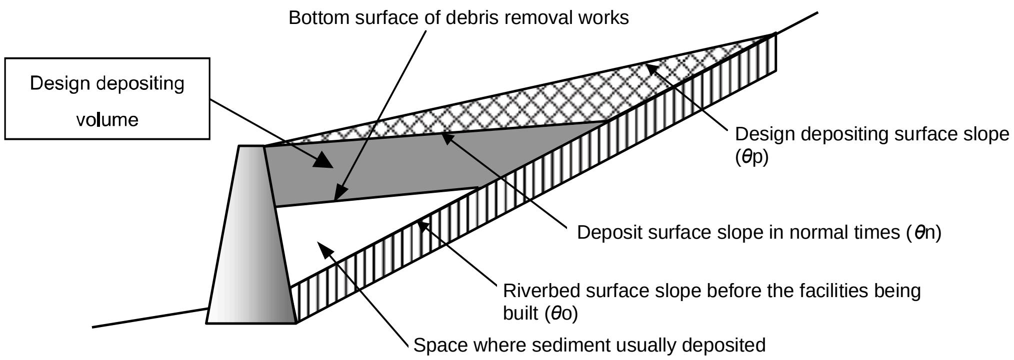


Figure 16 Concept of design depositing volume of debris flow and driftwood

3.3.1 Design sediment depositing volume

> The design sediment depositing volume is the sediment volume deposited upstream a debris flow and driftwood countermeasure facility in the volume of the “debris flow with design scale” and contained driftwood.

Explanation

The design sediment depositing volume is the sediment volume that shall be deposited in the space equivalent to the design depositing volume (gray colored in Figure 16).

3.3.2 Design driftwood depositing volume

The design driftwood depositing volume is the volume of driftwood deposited upstream a debris flow and driftwood countermeasure facility in the volume of the "debris flow with design scale" and contained driftwood.

Explanation

The method to calculate the design driftwood depositing volume is the same with the method to calculate the design driftwood retention volume in Section 3.2.2. It is specified as follows:

(1) The design driftwood depositing volume for semi-open type Sabo dam

The design driftwood depositing volume for semi-open type Sabo dam is calculated using Equation (36).

$$Y_{w1} = K_{w12} \times Y \quad \text{.......} (36)$$

Where, Y: design depositing volume of the debris flow material and driftwood (m^3), Y_{w1}: design driftwood depositing volume of the main dam (m^3), K_{w12}: ratio of driftwood content to design depositing volume of the debris flow material and driftwood. The value of K_{w12} is similarly set as K_{w11} is set as explained in Section 3.2.2 (1).

(2) The design driftwood depositing volume for closed type Sabo dam

The design driftwood depositing volume for closed type Sabo dam is the smaller value between the calculation results of Equation (37-1) and Equation (37-2). This method is similar to the calculation of design driftwood retention volume in Section 3.2.2 (2).

$$Y_{w1} = K_{w0} \times Y \times (1-\alpha) \cdot \cdot \cdot \cdot \cdot \cdot \cdot (37\text{-}1)$$

$$Y_{w1} = K_{w12} \times Y \times (1-\alpha) \cdot \cdot \cdot \cdot \cdot \cdot \cdot (37\text{-}2)$$

Where, Y: design depositing volume of debris flow material and driftwood (m^3), Y_{w1}: design driftwood depositing volume of the main dam (m^3), α : ratio of driftwood volume overflowed Sabo dam, K_{w0}: ratio of driftwood content to the design volume of debris flow material and driftwood which is expected to flow into the dam, K_{w12}: ratio of driftwood content to the design depositing volume of debris flow material and driftwood. The values of α and K_{w12} are similarly set as α and K_{w11} are set as explained in Section 3.2.2 (2).

3.4 Design volume of debris flow and driftwood prevention

The design volume of debris flow and driftwood prevention is the volume not to be entrained by the "debris flow with design scale" and contained driftwood, prevented by a debris flow and driftwood countermeasure facility. The design volume of debris flow and driftwood prevention is the total of the design sediment prevention volume and the design driftwood prevention volume.

Explanation

The design volume of debris flow and driftwood prevention is set for volume of entrainable channel deposit, volume of sediment prone to collapse and volume of driftwood in the zone where the design volume of debris flow material and driftwood is evaluated.

3.4.1 Design sediment prevention volume

The design sediment prevention volume is the sediment volume not to be entrained by the "debris flow with design scale", prevented by debris flow and driftwood countermeasure facilities.

Explanation

If any entrainable channel deposits exists, the design sediment prevention volume is calculated from the area between the intersection of the surface of design depositing surface and the surface of riverbed before the facilities being built and the location of Sabo dam (area with vertical pattern in Figure 17 (1) and (2)).

- For debris flow prevention work

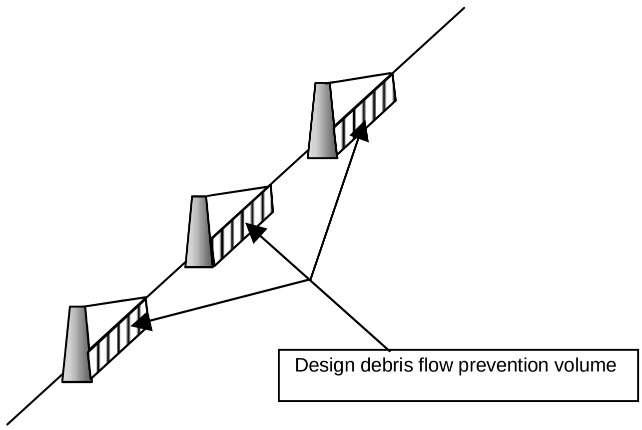
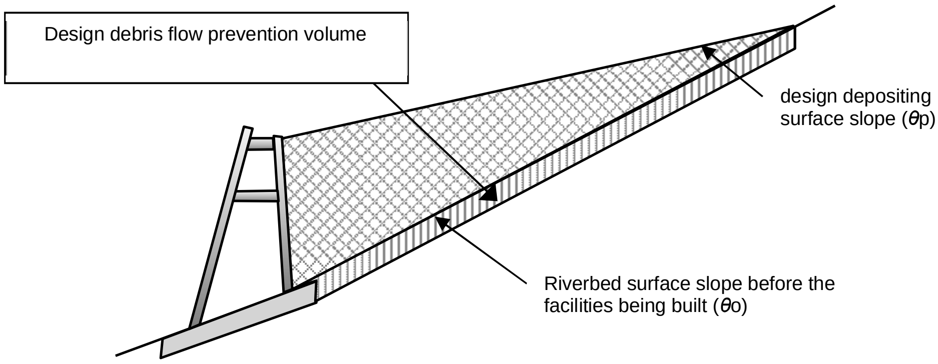


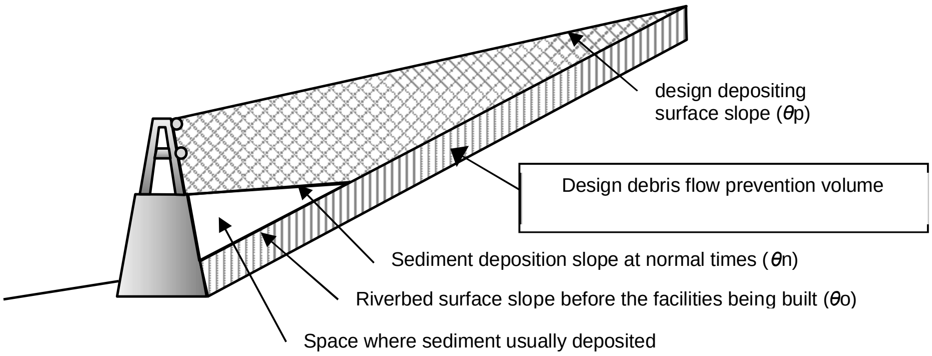
Figure 17 (1) Concept of design debris flow prevention volume

• For debris flow and driftwood capturing work

Open type



Semi-open type



Closed type

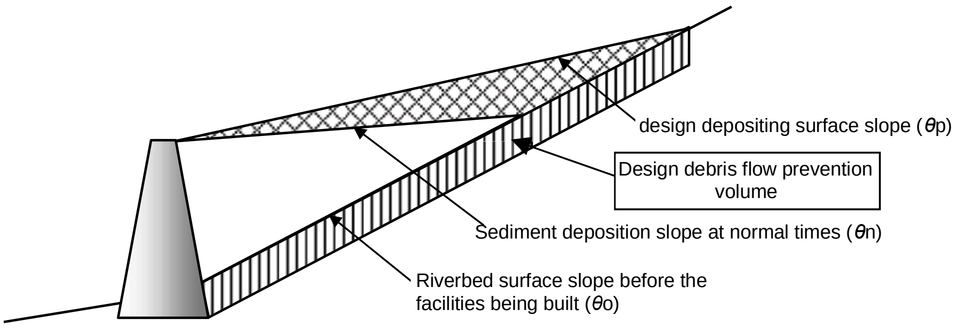


Figure 17 (2) Concept of design sediment prevention volume

3.4.2 Design driftwood prevention volume

The design driftwood prevention volume is the driftwood volume not to be recruited by the “debris flow with design scale”, prevented by debris flow and driftwood countermeasure facilities.

Explanation

The design driftwood prevention volume is set from the volume of driftwood in the zone where the design volume of driftwood is evaluated. The design driftwood prevention volume is equal to the deposited fallen trees and driftwood volume in the section between the intersection of the surface of riverbed before the facilities being built and the surface of deposit in normal times and the location of Sabo dam.

Section 4. Spatial distribution plan of facilities against debris flow and driftwood

4.1 General principles

The debris flow and driftwood countermeasure facilities shall be spatially arranged to satisfy the design capturing volume of debris flow material and driftwood, design depositing volume of debris flow material and driftwood, and design volume of debris flow and driftwood prevention that were set in the debris flow and driftwood control plan.

Explanation

The debris flow and driftwood countermeasure facilities shall be spatially arranged to handle the amount of sediment and driftwood in the plan. Additionally, their impact on the natural environment and landscape shall be fully considered.

4.2 Principles of spatial distribution plan of facilities against debris flow and driftwood

> The spatial distribution of debris flow and driftwood countermeasure facilities shall be arranged so that they can rationally and effectively control debris flow and driftwood. Such spatial arrangement shall consider the design volumes of sediment and driftwood, sediment transport pattern, position of the protected area, etc. Debris flow and driftwood capturing work is the main countermeasure facility against debris flow and driftwood.

Explanation

Debris flow and driftwood capturing work, debris flow depositing work, debris flow torrent training work, and debris flow and driftwood prevention work are combined to determine the locations, height of the facilities, and other features of the facilities. Further, debris flow and driftwood capturing works is the main facilities against debris flow and driftwood. However, if the target basin is devastated, debris flow and driftwood prevention works shall be implemented appropriately.

This concept is identical with the application in non-volcanic and volcanic mountains. However, countermeasures are often difficult to be implemented in volcanic mountains particularly during volcanic activities. Countermeasure plan for volcanic mountains shall be established by considering the occurrence of large failures and large mudflows.

Further, the erosion control greenbelt, debris flow direction-controlling work, and debris flow torrent training work shall be considered by taking into account the land use of volcanic area, particularly during the volcanic activity.

4.3 Functions and locations of debris flow and driftwood countermeasure facilities

Debris flow and driftwood countermeasure facilities consist of (1) debris flow and driftwood capturing work, (2) debris flow and driftwood prevention work, (3) debris flow torrent training work, (4) debris flow depositing work, (5) erosion control greenbelt, and (6) debris flow direction-controlling work.

Explanation

The regular type of debris flow and driftwood countermeasure facilities is (1) the debris flow and driftwood capturing work.

Other facilities are (2) debris flow and driftwood prevention work, (3) debris flow torrent training work, (4) debris flow depositing work, (5) erosion control greenbelt, (6) debris flow direction-controlling work, etc.

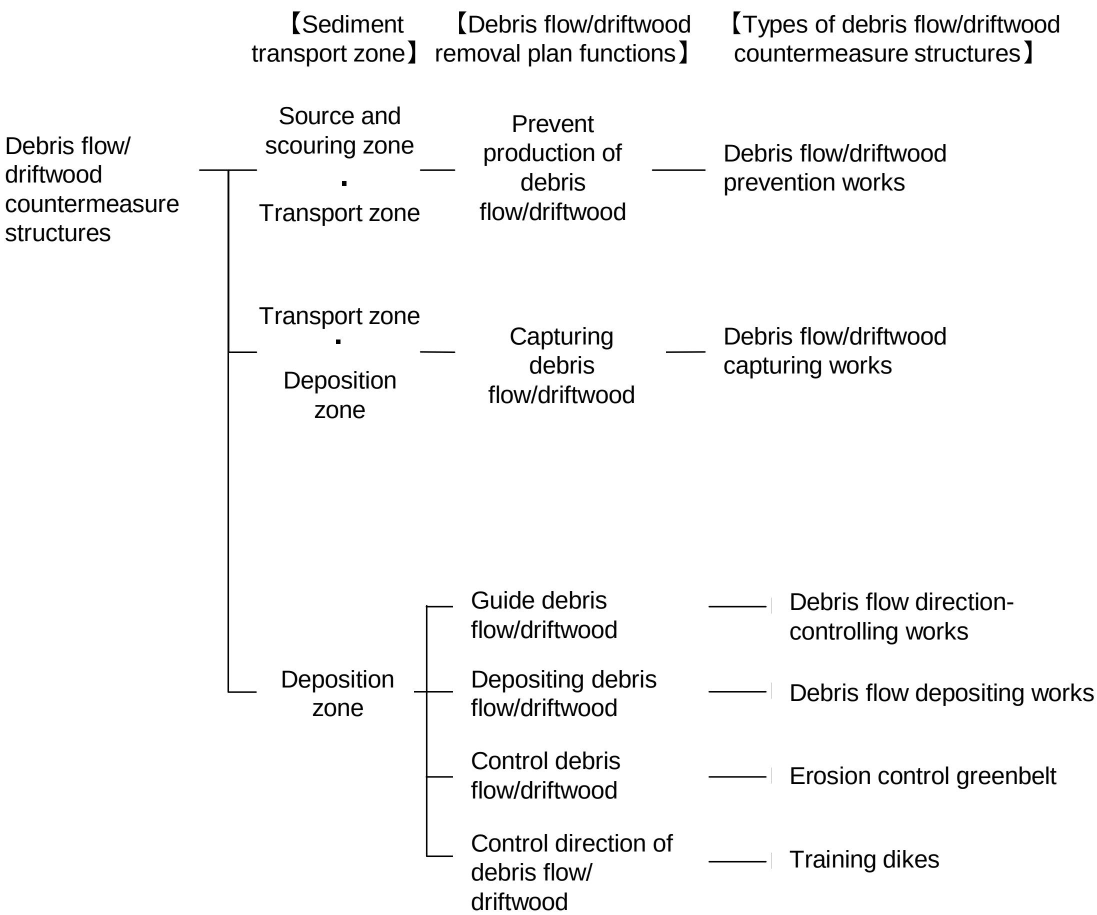


Figure 18 Types of debris flow and driftwood countermeasure facilities

4.3.1 Debris flow and driftwood capturing work

> Debris flow and driftwood capturing work is a debris flow and driftwood countermeasure facility aimed to capture debris flow and driftwood. Sabo dams shall be used as a debris flow and driftwood capturing work.

Explanation

When planning and arranging the debris flow and driftwood capturing work, the type and shape shall be determined by assuming the expected discharge of debris flow and driftwood as well as the grain size of the sediment, concentration of debris flow, size of driftwood (length and thickness), and the number of driftwoods. In addition, it shall be recognized that the sediment transport pattern of debris flow changes when the deposit surface slope in normal times changed significantly from riverbed surface slope before the facilities being built, or when the depositing area of the Sabo dam is long.

Sabo dam is mainly used as the debris flow and driftwood capturing work. However, debris flow breaker (drainage screen) is also considered as a debris flow and driftwood capturing work. This Guideline allows the application of debris flow and driftwood retention works other than Sabo dam.

4.3.1.1 Design volumes of sediment and driftwood controlled by Sabo dam

> The types of Sabo dams are open type, semi-open type and closed type. Depending on the dam type, the design volumes of sediment and driftwood which is expected for Sabo dams include design capturing volume of debris flow material and driftwood, design depositing volume of debris flow material and driftwood, and design volume of debris flow and driftwood prevention.

Explanation

The design volumes of sediment and driftwood of Sabo dams include design capturing volume of debris flow material and driftwood, design depositing volume of debris flow material and driftwood, and design volume of debris flow and driftwood prevention as shown in Figure 19 (1) and (2).

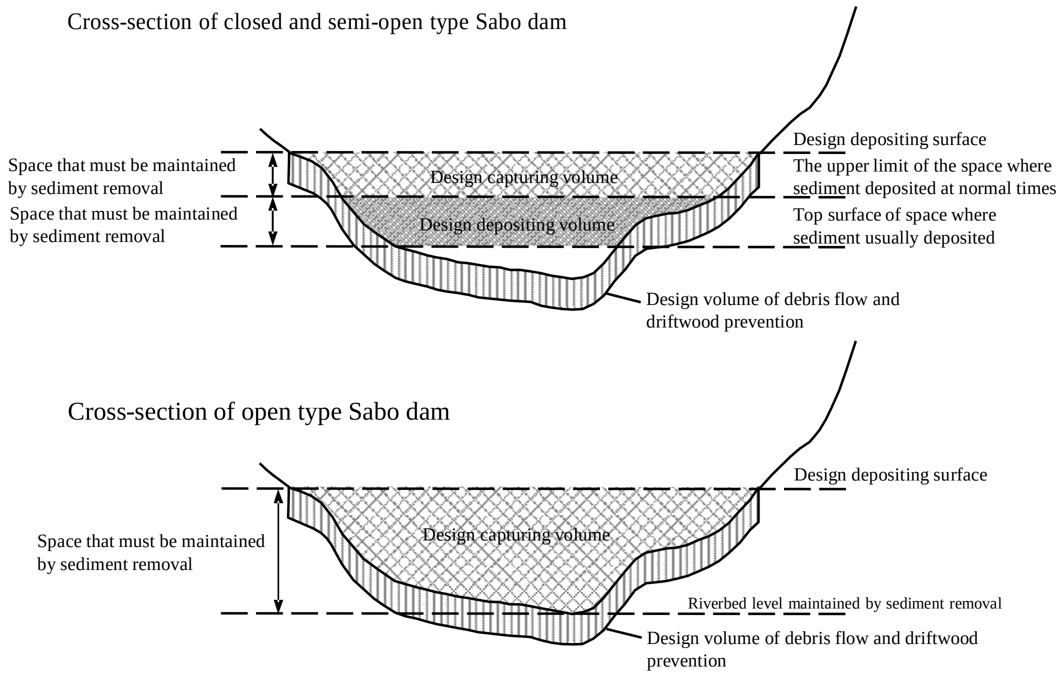
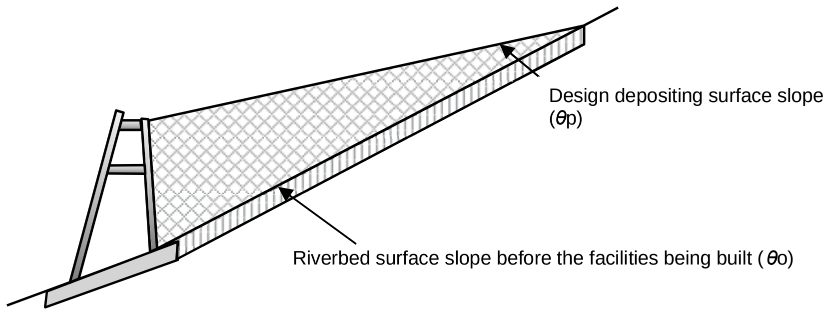


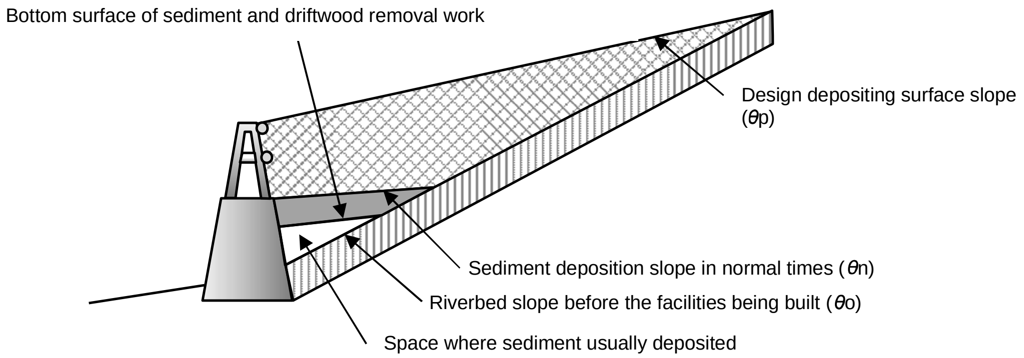
Figure 19 (1) Volume of sediment and driftwood depending on the type of Sabo dam

- Open type

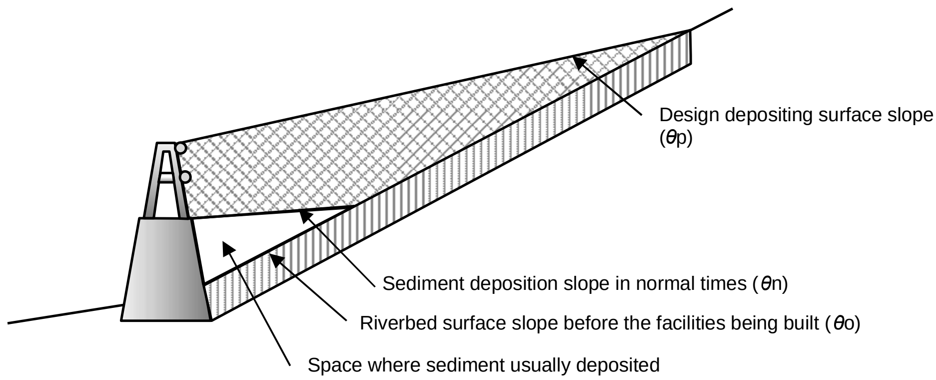


- Semi-open type

(If the space for design depositing volume can be ensured)

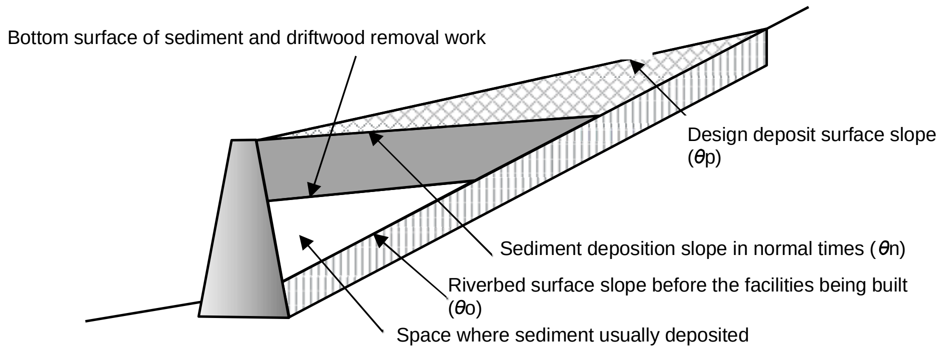


(If the space for design depositing volume cannot be ensured)



- Closed type

(If the space for design depositing volume can be ensured)



(If the space for the design depositing volume cannot be ensured)

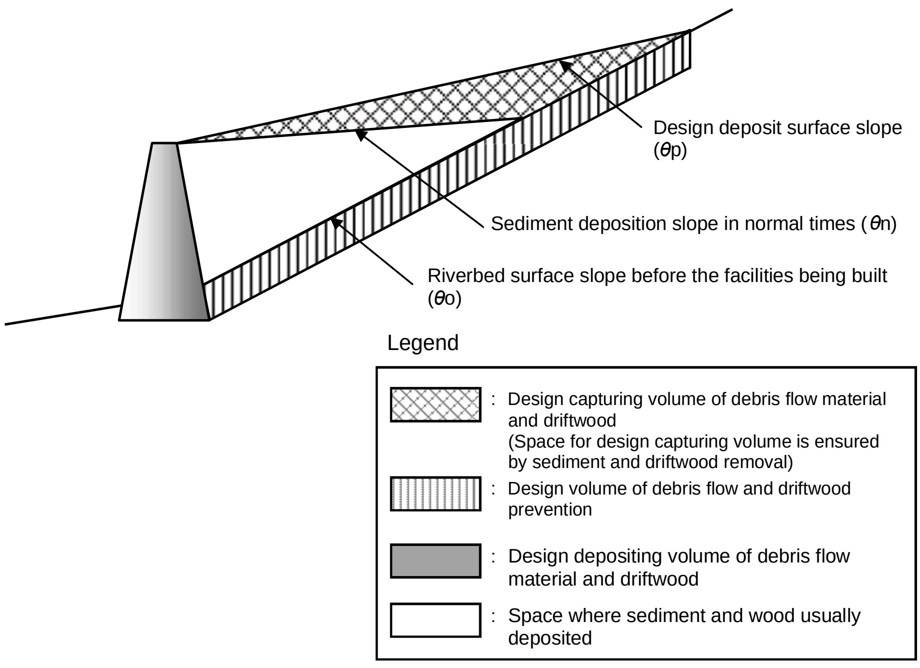


Figure 19 (2) Volumes of sediment and driftwood depending on the type of Sabo dam

4.3.1.2 Selection of the type of Sabo dam (open type, semi-open type, closed type)

The type of Sabo dam shall be examined from field survey result and considering the characteristics of the target basin, the expected event, the economic efficiency and the site's environment. In principle, structures with open section is required to capture all driftwood that flows with the sediment.

Explanation

Sabo dam is constructed in the source and scouring zone with the main function of preventing the occurrence of debris flows and driftwood.

Meanwhile Sabo dam constructed in transport zone or deposition zone must primarily provide the following functions:

- Capturing debris flow
- Capturing driftwood
- Maintain the space corresponding to the design capturing volume of debris flow and driftwood
- Conserve river environment at normal times (continuity of the river)

Open-type facilities (e.g., open type Sabo dam, semi-open type Sabo dam, driftwood retention work, etc.) are required to capture all driftwood that flows with the sediment. Therefore, if the design allowable volume of driftwood is not zero or driftwood countermeasure facilities are planned separately, open or semi-open type Sabo dam shall be selected to capture/retain driftwood. Further, if driftwood retention work needs to be installed in the debris flow zone, it can be installed at the sub-dam.

In addition, sediment and driftwood removal plan needs to be considered to keep empty the design capturing volume of debris flow and driftwood regardless the dam type. For closed and semi-open type Sabo dams where design depositing volume of debris flow material and driftwood is planned, sediment and driftwood removal plan to keep empty the design depositing volume of debris flow material and driftwood need to be examined. The sediment and driftwood removal plan is explained in Section 5 of the Guideline.

4.3.1.3 Types and spatial distributions of open and semi-open Sabo dam

Open or semi-open type Sabo dam shall be designed to ensure that the open section shall be fully blocked by large stones, boulders, etc. so that the dam shall be capturing the designed scale of debris flow. In addition, the sediment transport pattern shall be considered when arranging the spatial distribution of open or semi-open type Sabo dam.

Explanation

(1) Basic concept on the spatial distribution of open and semi-open type Sabo dams

Open and semi-open type Sabo dams capture debris flow as the boulders carried by the debris flow block its open section. In addition, if the open section is fully blocked, the risk of captured sediment to flow downstream is almost non-existent. For this reason, open and semi-open type Sabo dams are usually situated in the debris flow zone.

On the other hand, open and semi-open type Sabo dams with the purpose of raising backwater and temporarily depositing sediment shall not be constructed in debris flow zone. It is due to the risk of accumulated sediment flowing downstream during the recession stage of flood.

(2) Precautions when arranging the spatial distribution and designing Sabo dam to capture debris flow

Open and semi-open type Sabo dams must satisfy the following conditions to capture debris flow.

① The open section shall be fully blocked by the “debris flow with design scale” and contained driftwood, but the structure shall not be destroyed during the debris flow.

If open or semi-open type Sabo dam is planned in the deposition zone, it shall be arranged considering the change of sediment transport pattern so that the entire open section is blocked by boulder and driftwood. If multiple Sabo dams are planned, a change in sediment transport pattern due to the open type Sabo dams located at the river upstream shall be considered.

② The open section shall not be blocked by bed load sediment transported by the flow during low to moderate rainfall.

Open type Sabo dam is expected to capture the design capturing volume of debris flow and driftwood without any sediment deposition during small or medium-sized flood. However, similar to closed type, open and semi-open type must be maintained by sediment and driftwood removal for example, after a debris flow has been captured.

Steel pipes or concrete used to construct the open section are categorized to those that maintain the stability of the structure (structural members) and those installed to capture the debris flow (functional members). Plastic deformation is allowed for the functional members as long as they can capture debris flow and driftwoods.

Furthermore, semi-open type can be adopted for the following cases:

- Driftwood capturing function needs to be enhanced beside prevention of debris flow and driftwood is required.
- It is expected that fine particle of sediment is dominant.
- Low sediment concentration is expected due to gentle slope.
- Muddy water due to flooding (other than debris flow) near the mouth of valley needs to be directed to downstream channel.

4.3.2 Debris flow and driftwood prevention work

Debris flow and driftwood prevention work is a debris flow and driftwood facility that prevents debris flows and driftwood.

Explanation

Debris flow and driftwood prevention work includes the restraint work on hillslopes, prevention works on riverbed or on riverbank.

4.3.2.1 Hillslope restraint work

A hillslope restraint work such as vegetation or other facilities shall be used to stabilize mountain slopes.

Explanation

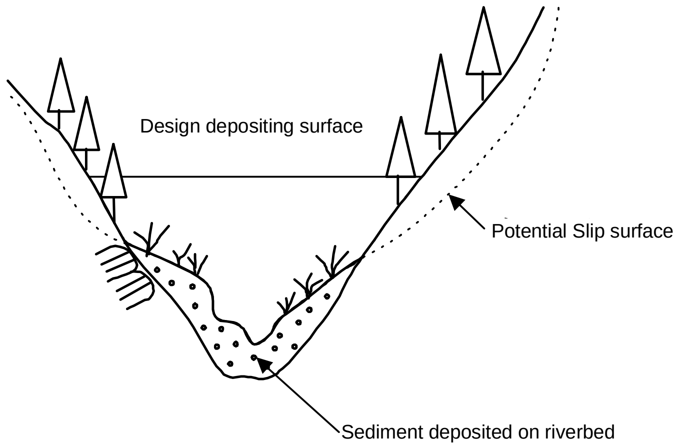
Hillslope restraint works are conducted to prevent the collapse of hillslopes that may cause debris flow and driftwood.

4.3.2.2 Riverbed sediment stabilization work

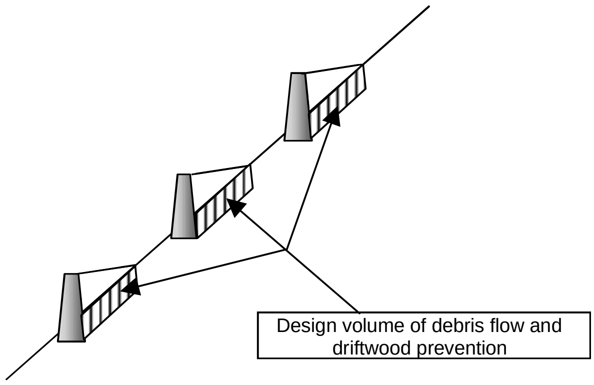
A riverbed sediment stabilization work shall prevent the collapse of river banks and the movement of sediments deposited on the riverbed by consolidation work or other methods.

Explanation

In order to prevent the movement of sediment deposited on riverbed and the collapse of river banks, consolidation work, revetment work for embankment, etc. are considered. To prevent the collapse of river banks (including hillslopes), sediment and driftwood removal shall not be performed for the riverbed sediment stabilization work.



Lateral cross-section



Longitudinal cross-section

Figure 20 Image of sediment and driftwood volume in the riverbed sediment stabilization work

4.3.3 Debris flow torrent training work

A debris flow torrent training work shall direct debris flow to a safe place, and shall have a cross section through which the peak discharge of debris flow for this section can flow down.

Explanation

Debris flow torrent training work must carefully consider the grain size of the sediment so that deposition does not occur inside the debris flow torrent training work, because deposition inside the debris flow torrent training work should cause overflow and inundation.

If there have been some facilities upstream, the capturing and/or depositing effects shall be considered to reduce the peak discharge.

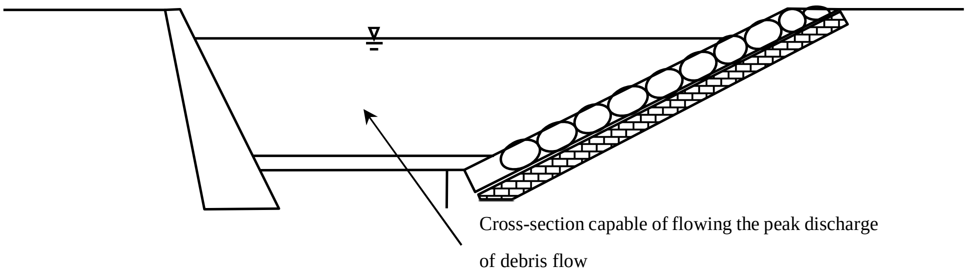


Figure 21 Debris flow torrent training work

4.3.4 Debris flow depositing work

Debris flow depositing works are the debris flow and driftwood countermeasure facilities that dissipate and deposit debris flows. Such structures include debris flow dispersion/depositing area and debris flow depositing channels.

Explanation

Debris flow depositing works shall deposit debris flows safely, and consist of two types: debris flow dispersion/depositing area and debris flow depositing channels.

(1) Debris flow dispersing/depositing area

A debris flow dispersing/depositing area is a land created by widening a channel where Sabo dams or groundsills are constructed at its upstream and downstream ends.

A debris flow dispersing/depositing area ensures a space to deposit the debris flow material and contained driftwood with the design depositing volume of debris flow and driftwood. Such space is secured by widening the channel and excavating its bed to reduce the bed surface slope.

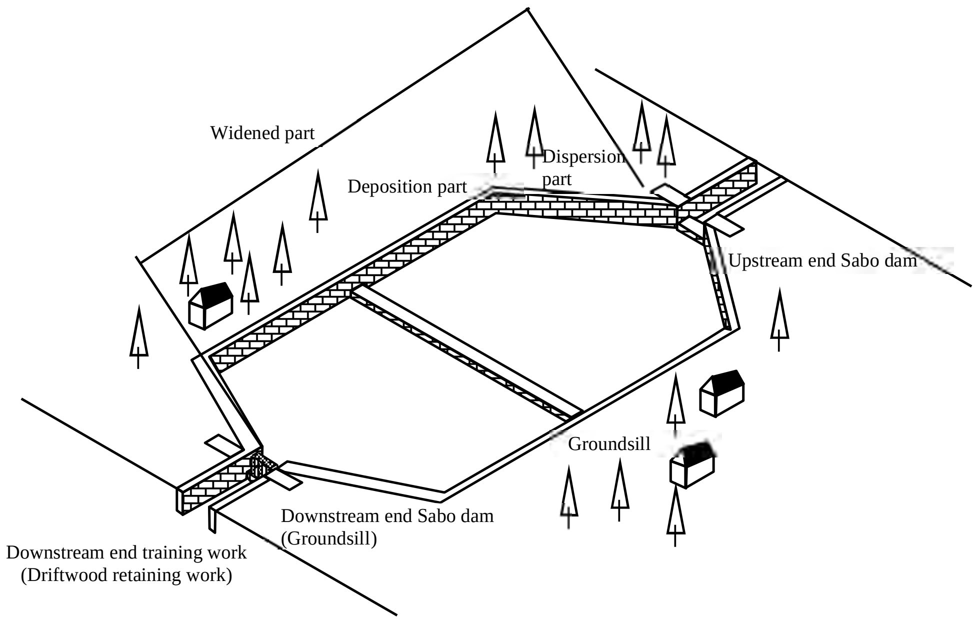


Figure 22 Debris flow dispersing/depositing area

(2) Debris flow depositing channel

A debris flow depositing channel is constructed to ensure a space to deposit the debris flow material and contained driftwood with the design depositing volume of debris flow and driftwood. Such space is created by excavating the channel to lower the riverbed slope. This method is used when a debris flow dispersion/depositing area is difficult to construct due to the development of residential land in the surrounding areas or due to the topographical conditions of the valley bottom plain.

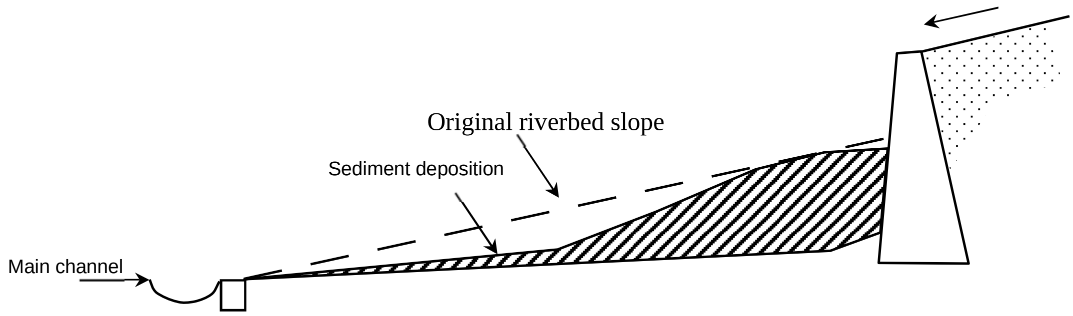


Figure 23 Debris flow depositing channel

4.3.5 Erosion control greenbelt

An erosion control greenbelt is a debris flow and driftwood countermeasures which aims to reduce the flow velocity and deposit the debris flow.

Explanation

As an erosion control greenbelt, debris flow and driftwood countermeasures such as consolidation work and debris flow direction-controlling work are combined with forests, main channel section, and other supplementary structures. Such combination is constructed near the downstream end of the deposition zone.

In principle, erosion control greenbelt is combined with debris flow direction-controlling works as a buffer between debris flows and properties to be protected on an alluvial fan land.

4.3.6 Debris flow direction-controlling works

A debris flow direction-controlling work is a debris flow and driftwood countermeasure work that controls the direction of a debris flow.

Explanation

If safe places to flow sediment downstream from the reference point are available, and the flowing process shall not cause any damage or disaster, the direction of the debris flow can be controlled by debris flow direction-controlling works.

Section 5. Sediment and driftwood removal plan

> To maintain the debris flow and driftwood facility's full function, sediment deposition shall be inspected regularly after the occurrence of debris flow and sediment and driftwood removal shall be conducted as necessary.
>
> Further, if debris flow and driftwood control plan requires sediment and driftwood removal work, the way of transporting the sediment including a route shall be sought in advance.

Explanation

If debris flow and driftwood control plan requires sediment and driftwood removal work, the plan for sediment and driftwood removal shall be examined in the debris flow and driftwood control plan. The plan for sediment and driftwood removal includes methods of transportation, route for carrying out, destination of the removed sediment, frequency of sediment and driftwood removal, etc. In principle, sediment and driftwood removal is not conducted for riverbed sediment stabilization work.

Sediment and driftwood removal are classified to emergency removal which is conducted as an emergency measure after a debris flow event and periodic removal which is conducted based on regular inspections. The basic concepts are described below.

(1) Emergency sediment and driftwood removal

Urgent removal of sediment and driftwood transported by a debris flow is important to ensure the design capturing volume and the design depositing volume of debris flow and driftwood of a Sabo dam.

For this reason, an exceptional inspection shall be conducted to check the capturing status of the debris flow and driftwood countermeasure facility. If necessary, an emergency sediment and driftwood removal is performed in preparation for the next future debris flow.

(2) Periodic sediment and driftwood removal based on regular inspections

Periodic removal based on regular inspections is performed mainly to ensure the design depositing volume of debris flow and driftwood from the accumulation of sediment and driftwood.

As a result of the periodic inspections, sediment and driftwood removal shall be conducted if the design capturing volume and design depositing volume of debris flow and driftwood need to be empty.

Conducting the sediment and driftwood removal, removal shall not be started just upstream of the facility but be started from the upstream end of deposited area. This is to avoid the possible danger of an abrupt outburst of captured sediment and driftwood.

References

1) Ministry of Construction, River Bureau, Sabo Department Sabo Division (1999): Debris flow-prone river and debris flow risk district survey methods (tentative), p. 17
2) W. Sakurai (2002): Research on volume of transported sediment by a debris flow produced on hillslopes, Civil Engineering Journal, 44-4, p 6-7
3) T. Kudo, T. Uchida, N. Matsumoto, W. Sakurai(2015): Analysis of Eroded Width and Depth Due to Debris Flow using LiDAR Data, Civil Engineering Journal, 57-11, p 22-25
4) N. Osanai, T. Uchida, M. Sokabe, H. Terada, K. Kondou(2005): Research on a method of estimating the potential depth of slope failure using a knocking pole test, Technical Note of National Institute for Land and Infrastructure Management, No.225, pp.46
5) Sabo Division, Sabo Department, River Bureau, Ministry of Construction(1989): *Debris flow countermeasure technology guideline (tentative)*
6) A. Matsuoka, T. Yamakoshi, K. Tamura, Y. Nagai, J. Maruyama, T. Kotake, K. Ogawa, S. Tagata(2009): Sediment dynamics in the mountainous watersheds based on differentiation of multi-temporal LiDAR survey data sets, Journal of the Japan Society of Erosion Control Engineering, Vol. 62, No. 1, p.60 – 65
7) Y. Ishikawa, T. Mizuyama, M. Fukusawa (1989): Generation and flow mechanisms of floating logs associated with debris flow, Journal of the Japan Society of Erosion Control Engineering, Vol. 42, No. 3, p.4-9
8) T. Kanno, T. Yoshida, T. Ishida, Y. Miyahara, T. Fuzimoto, T. Moriiwa(2010): How to utilize aviation LiDAR measurement data for forest phase classification and wood volume calculation for driftwood volume calculation, Abstracts of 2010 JSECE symposium, p.131-132
9) I. Mine (1958): Forest mensuration, p. 146, Asakura Publishing,
10) T. Mizuyama (1990): Empirical prediction of peak discharge of debris flow, priority field research of the Ministry of Education, Science and Technology Fund, “Predicting natural disasters and prevention of disasters”, Research on predicting the occurrence and scale of debris flows, priority field research of the Ministry of Education, Science and Technology Fund, “Predicting natural disasters and prevention of disasters”, p. 54
11) T. Takahashi (1978): The occurrence and flow mechanism of debris flow, *Tsuchi to kiso* (Soil and Grounds), Vol. 26, No. 6, p. 46
12) K. Ashida, T. Takahashi, T. Sawada (1976): Runoff process, sediment yield and transport in a mountain watershed, DPRI Annuals, 19-B, p. 345
13) T. Mizuyama, K. Senoo (1984): Flood traveling time in small mountain basins and rainfall intensity in short duration, Journal of the Japan Society of Erosion Control Engineering, Vol. 37, No. 3, p. 20 and Corrected report in Journal of the Japan Society of Erosion Control Engineering, Vol. 39, No. 1, p. 16
14) S. Suematsu eds. (1954): River engineering handbook, Vol. 1, p. 64, Corona publishing
15) T. Mizuyama, S. Uehara (1984): Observed data of the depth and velocity of debris flow, Journal of the

Japan Society of Erosion Control Engineering, Vol. 37, No. 4, p. 23

16) T. Mizuyama, S. Uehara (1981): Debris flow in steep slope channel curves, Civil Engineering Journal, Vol.23, No.5, p 243-248

17) N. Takezawa, T. Uchida, R. Suzuki, K. Tamura: Estimation of debris flow induced by a deep-seated landslide at the Funaishi river basin, Kagoshima prefecture in Japan, Journal of the Japan Society of Erosion Control Engineering, Vol. 62, No. 2, p. 21-28

18) T. Osaka, E. Takahashi, M. Kunitomo, T. Yamakoshi, Y. Nowa, H. Kisa, T. Ishizuka, R. Utsunomiya, K. Yokoyama, T. Mizuyama(2013): Field observations of unit weight of flowing debris flows by force plate in Sakurazima, Japan, Journal of the Japan Society of Erosion Control Engineering, Vol. 65, No. 6, p. 46-50

19) N. Fujimura, T. Kuroiwa, H. Izumiyama, F. Akazawa, H. Mizuno(2016): Report on Experimental Analysis of Driftwood release and trapping by Closed type Sabo dam, Technical Note of PWRI, Vol.4331

Appendix E List of symbols

A	target basin area (km^2)
A_d	cross-sectional area of debris flow peak discharge (m^2)
A_{dy11}	average cross-sectional area of the entrainable channel deposits (m^2)
A_{dy12}	average cross-sectional area of the sediment entrainable by debris flow and prone to collapse in the zero-order stream (m^2)
A_w	plane area of the driftwood deposition or retention pond of the driftwood retention work (m^2)
B_d	average riverbed width where erosion is predicted to occur during debris flow (m)
B_{dm}	the minimum flow width of debris flow (m)
B_{da}	width of debris flow (m)
C_*	volumetric concentration of sediment deposited on the riverbed (approximately 0.6)
C_d	concentration of debris flow
D_{95}	grain size for which 95% of the material weight is finer
D_d	depth of the debris flow (m)
D_e	average depth of the riverbed sediment where erosion is predicted to occur during debris flow (m)
D_r	debris flow hydraulic radius (m)
F	drag force of debris flow per unit of width (kN/m)
g	gravitational acceleration (9.81 m/s^2)
H_w	tree height (m)
h_{wa}	average tree height (m)
H_{wm}	the maximum tree height (m)
K_d	breast height coefficient
K_{f1}	peak discharge coefficient
K_{f2}	discharge adjustment factor
K_h	drag coefficient (= 1.0)
K_n	roughness coefficient ($s \cdot m^{-1/3}$)
K_{p1}	coefficient for T_f
K_{p2}	coefficient for P_a
K_q	coefficient (= $C_* / (C_* - C_d)$)
K_{w0}	ratio of driftwood content to the design volume of debris flow and driftwood
K_{w11}	ratio of driftwood content to the design capturing volume of debris flow and driftwood
K_{w12}	ratio of driftwood to design depositing volume of the debris flow material and driftwood
K_v	porosity
L_{dy11}	river length from the reference point, supplementary reference point, or the point where the volume of debris flow material is to be calculated to the furthest upstream point of the first-order stream (m)
L_{dy12}	river length from highest point of the first-order stream where the volume of debris flow material is calculated to the farthest point in the target basin (m)
L_{dy13}	river length from the point where the driftwood yield is calculated to the farthest point in the target basin (m)
L_{wa}	average length of driftwood (m)
L_{wm}	the maximum length of driftwood (m)
P_{24}	24 - hour rainfall (mm)
P_a	average rainfall intensity during the flood concentration time (mm / h)
P_{day}	Daily rainfall (mm)
P_e	effective rainfall intensity (mm / h)
P_p	rainfall of the annual exceedance of probability (mm)
ΣQ	total sediment volume containing in a debris flow (m^3)
Q_p	peak discharge of water without sediment under rainfall with the design exceedance probability (m^3/s)
Q_{sp}	peak discharge of debris flow (m^3/s)
R_w	breast height diameter (m)
R_{wa}	average diameter of driftwood (m)
T_f	flood concentration time (min)
U	flow velocity of debris flow (m/s)
V	design volume of debris flow material and driftwood (m^3)

V_d	design volume of debris flow material (m^3)
V_{dqp}	sediment volume discharged by the largest surge of debris flow (including void) (m^3)
V_{dy1}	volume of sediment entrainable by debris flow and prone to collapse in the target basin (m^3)
V_{dy11}	volume of entrainable channel deposits in the section from the reference point, supplementary reference point, or the point where the volume of debris flow material is to be calculated to the furthest upstream point of the first-order stream (m^3)
V_{dy12}	volume of sediment prone to collapse (m^3)
V_w	design volume of driftwood (m^3)
V_{wc}	actual volume of the driftwood deposited on the depositing area or in the reservoir (m^3)
V_{wy}	volume of driftwood yield (m^3)
V_{wy1}	driftwood yield per unit area of target basin (m^3 / km^2)
V_{wy2}	single tree volume (m^3)
ΣV_{wy2}	tree volume per 100 m^2 in the sampling survey (m^3/100 m^2)
W	design allowable volume of debris flow material and driftwood
W_d	design allowable volume of debris flow material (m^3)
W_w	design allowable volume of driftwood (m^3)
X	design capturing volume of debris flow and driftwood by debris flow and driftwood facilities (m^3)
X_d	design sediment capturing volume (m^3)
X_w	design driftwood retention volume (m^3)
X_{w1}	design driftwood retention volume by Sabo dam (m^3)
X_{w2}	Design driftwood retention volume by a sub-dam (m^3)
Y	design depositing volume of the debris flow and driftwood countermeasure facilities (m^3)
Y_d	design sediment depositing volume (m^3)
Y_w	design driftwood depositing volume (m^3)
Y_{w1}	design driftwood depositing volume of the dam (m^3)
Z	design volume of debris flow and driftwood prevention (m^3)
Z_d	design sediment prevention volume (m^3)
Z_w	design driftwood prevention volume (m^3)
α	ratio of driftwood discharge from Sabo dam (approx. 0.5)
γ_d	debris flow unit weight (kN/m^3)
θ	riverbed surface slope (degrees)
θ_0	riverbed surface slope before the facilities being built (degrees)
θ_n	deposit surface slope in normal times (degrees)
θ_p	design deposit surface slope (degrees)
ρ	density of water (approx. 1,200 kg/m^3)
σ	density of gravel (approx. 2,600 kg/m^3)
ϕ	internal friction angle of sediment deposited on a riverbed (degrees) (approximately 30-40°; 35° is commonly used)

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No.905　　　　　　　　　　　May 2022

土石流・流木対策設計技術指針解説（英訳）

土砂災害研究部　砂防研究室

Technical Guideline for Designing Sabo Facilities against Debris Flow and Driftwood

Sabo Planning Division
Sabo Department

国土交通省　国土技術政策総合研究所

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概　要

本資料は、国総研資料第905号「土石流・流木対策設計技術指針解説」を英訳したものである。

キーワード：土石流、砂防設備、設計

Synopsis

This note is the translation of the technical note of NILIM No.905 (Manual of Technical Guideline for Designing Sabo facilities against Debris Flow and Driftwood, originally written in Japanese in April, 2016).

Keywords: Debris flow, Sabo facilities, design

Disclaimer

This is a tentative translation of the original written in Japanese in April, 2016. However, some portions are modified or are not translated in case that they seem to be impossible to understand for those who don't know the context of the Japanese system of the technology standard or the domestic situations in Japan.

If what is translated in English is found to be different from what is written in Japanese version, the latter is always true.

This tentative translation could be subject to be change without any prior notice.

Table of Contents

Section 1. General Principles

Debris flow and driftwood facilities shall be designed to have necessary functions and safety based on the Sabo master plan against debris flow and driftwood.

Explanation

The Technical Guideline for Designing Sabo Facilities against Debris Flows and Driftwood (hereafter called "the Guideline") explains the methods of designing debris flow and driftwood facilities stipulated by the Sabo master plan against debris flow and driftwood created based on *the Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood*.

The characteristics and condition of river varies by zone and changes over time. Thus, the spatial distribution and design of debris flow and driftwood facilities are prepared by clarifying functions suited to these characteristics. Such characteristics and condition are determined by field surveys and collection of documents regarding river characteristics including its changes over time.

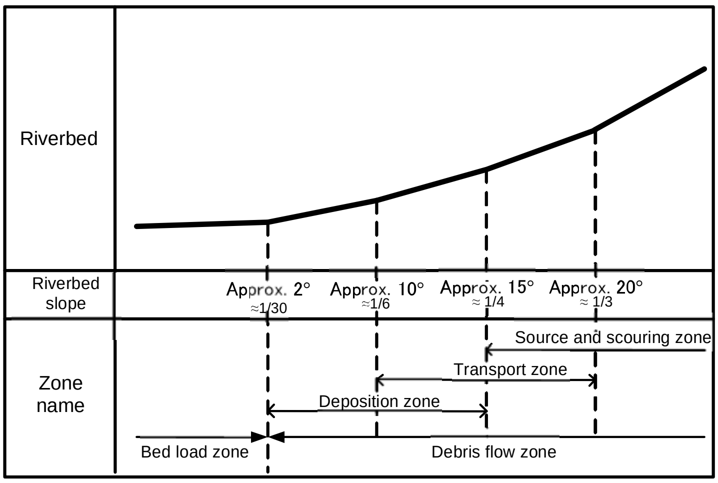

Figure 1 Changes of sediment transport pattern based on the riverbed slope [1)]

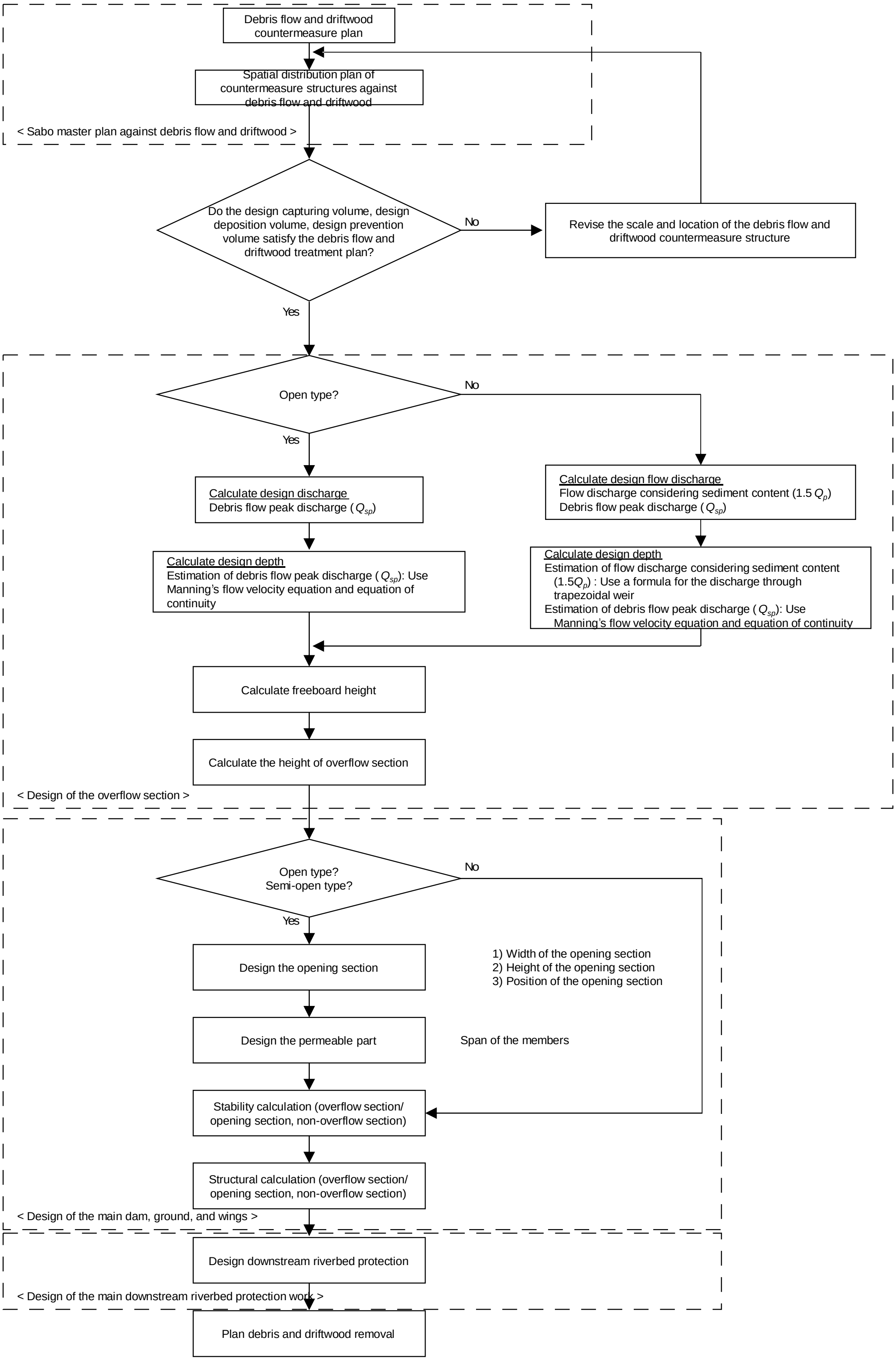

Figure 2 Flow chart of the debris flow and driftwood capturing work design

Section 2. Design of debris flow and driftwood facilities

2.1 Debris flow and driftwood capturing work

2.1.1 Type of debris flow and driftwood capturing work

The types of debris flow and driftwood capturing work include open, semi-open, and closed type.

Explanation

Sabo dams used as debris flow and driftwood capturing work are designed differently according to its type. Concerning functions of each type, see the explanation on *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* , Section 4.3.1.

2.1.2 Dimensioning and positioning of debris flow and driftwood capturing work

> The dimensioning and spatial distribution of a debris flow and driftwood capturing work shall be formulated based on Section 4 of the *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood*. However, the decision shall also be determined based on geological, topographical, and other site conditions.

Explanation

The dimensioning and spatial distribution of a debris flow and driftwood capturing work must be set based on the spatial distribution plan of facilities against debris flow and driftwood established under Section 4 of the *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood*. When designing the structure, dimensioning and spatial distribution of the debris flow and driftwood capturing work can be reviewed if necessary, and revised accordingly.

The location of debris flow and driftwood capturing work is selected appropriately considering the topography, geology, etc. If a curved section of the river must be selected, direction of the main body axis and protection works for the downstream riverbed shall be examined by considering the flow directions upstream and downstream of the debris flow and driftwood capturing work.

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(Appendix A) Design of Sabo facilities in a small ephemeral river

Sabo dam set in a small ephemeral river which has no branching channel need to be designed properly by fully considering the field conditions such as topography and geology. For the definition of "small ephemeral river", refer to the *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood*, Section 2.5.1.1. Note that there are cases[2)] where countermeasures in a small ephemeral river were examined and the following viewpoints may be referred to.

- The crest width shall be determined considering the riverbed materials, patterns of mass movement and sediment transport, target flow rate, etc. at the planned site. The crest width shall be twice the maximum boulder size that collides with the dam, but shall be at least 1.5 m.
- The slope of the top of the dam's wing is basically horizontal or steeper.
- Apron length shall be longer than the nappe distance which is calculated using semi-theoretical equation. This is to secure safety of main dam body from erosion on its downstream due to debris flow.

〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜〜

2.1.3 Structure of closed type Sabo dam

2.1.3.1 Stability of overflow section

> The entire closed type Sabo dam must be stable against overturning, sliding, and sinking. Further, members that configure the dam body must be safe against debris flow and driftwood.

Explanation

Stability of overflow section is calculated based on the method in the Guideline, Section 2.1.3.1(1).

Dam body shall be a safe structure based on the method in the Guideline, Section 2.1.3.2 (2) and (3) and stability of non-overflow section shall be also ensured as stated in Section 2.1.3.3 (1). The dam body shall be designed so that the dam body consisting of several types of members is integrally resisting the design external forces. In addition, when sediment is used as filling material and water flows constantly (e.g., a large basin), the dam should be designed to secure redundancy. For instance, using soil cement to solidify the sediment filling material, hence partial damage may not expand to the whole body.

(1) Stability conditions

> A closed type Sabo dam used as a debris flow and driftwood capturing work shall satisfy the following three conditions in order to maintain stability under the external forces in (2).
>
> 1. In principle, the resultant force consisting of the dam's self-weight and external forces must work within the range of the middle up to 1/3 of the dam base section. Hence, tensile stress is not generated at the upstream end of the Sabo dam.
> 2. Sliding shall not occur between the bottom of a Sabo dam and the ground.
> 3. The maximum stress generated inside the Sabo dam shall not exceed the allowable stress of the material. The maximum pressure received by the ground shall be within the allowable bearing capacity of the ground.

Explanation

For rock ground, the safety factor (N) against sliding is N = 4.0 considering the shear strength (the smaller value between the shear strength in the dam body and in the ground). While for sand and gravel ground, the shear strength is ignored and the safety factor is N = 1.2 for dam height of less than 15 m. While for dam height of 15 m or higher, the safety factor is N = 1.5.

(2) Design external forces

Design external forces considered in designing a closed type Sabo dam are the hydrostatic pressure, earth pressure, uplift pressure, inertia force during earthquakes, hydrodynamic pressure during earthquakes, and "load due to debris flows and driftwood" (hereinafter referred as "load of debris flow"). Load of debris flow includes drag force produced by a debris flow (hereinafter referred as "drag force of debris flow") and the force produced by the collision of boulder and driftwood. The former is considered to affect the entire structure, while the latter is considered to affect locally. Therefore, stability calculations for a Sabo dam shall deal with the drag force of debris flow only, while the force produced by the impact of boulder and driftwood shall be considered for the design of the crest width.

Explanation

In addition to the combinations of design external forces (in normal times and during flooding) presented in the *Technical Criteria for River Works (Tentative)* , Design, II, Chapter 3, Section 2.2.1, the following stability calculations during debris flows are performed and stability conditions must be satisfied by all combinations.

Combinations of design external forces are as shown in Table 1 in addition to the self-weight of a Sabo dam. "Design external forces (in normal times and during flooding)" referred in the Guideline are based on the "Loads used for Stability Calculations" in the *Technical Criteria for River Works (Tentative)* , Design, II, Chapter 3.

However, the design external forces for Sabo dam with height of less than 15 m is calculated by assuming the unit weight of the water as 11.77 kN/m^3.

During a debris flow, the most critical condition for the load of debris flow is assumed, i.e. a debris flow directly hits the main body while sedimentation proceeds until the remaining deposition height is same as the debris flow depth (D_d) (see Figure 3).

The drag force of debris flow acts horizontally at the position of D_d / 2. The deposited earth pressure consists of weight of debris flow as loading load, $C_e(\gamma_d - \gamma_W)D_d$, as well as deposited earth pressure up to the sedimentation surface.

Where,

C_e : coefficient of earth pressure,

D_d : depth of debris flow calculated using the current riverbed gradient (m),

γ_d : unit weight of the debris flow (kN/m^3),

γ_s : unit weight of sediment in water (kN/m^3),

γ_W : unit weight of water (γ_W is approximately 11.77 kN/m^3 for Sabo dam height of less than 15 m, and 9.81 kN/m^3 for Sabo dam height of 15 m or more).

$$\gamma_s = C_* \left(\sigma - \rho\right) g \quad \text{.................................(1)}$$

$\gamma_w = \rho g$ ....(2)

Where,

C_* : volumetric concentration of riverbed sediment,

ρ : water density (kg/m^3),

σ : gravel density (kg/m^3),

g : gravitational acceleration (m/s^2) (9.81 m/s^2).

The hydrostatic pressure during a debris flow acts only below the sedimentation surface because the drag force of debris flow acts above the sedimentation surface.

Table 1 Design external forces used for stability calculations for closed type dam (excluding self-weight)

	Normal time	During debris flow	During flooding
Dam height of less than 15 m		Hydrostatic pressure, earth pressure, drag force of debris flow	Hydrostatic pressure
Dam height of 15 m or more	Hydrostatic pressure, earth pressure, uplift pressure, inertia force during an earthquake, hydrodynamic pressure during an earthquake	Hydrostatic pressure, earth pressure, uplift pressure, drag force of debris flow	Hydrostatic pressure, earth pressure, uplift pressure

* According to the past large earthquakes experiences such as Kobe Earthquake 1995, Sabo dams with height of less than 15 m have never been severely damaged, lost their functions, or cause direct and secondary damages to the surrounding buildings. The results of dynamic analysis have shown that they are safe from tensile stress, compressive stress, and sliding [3].

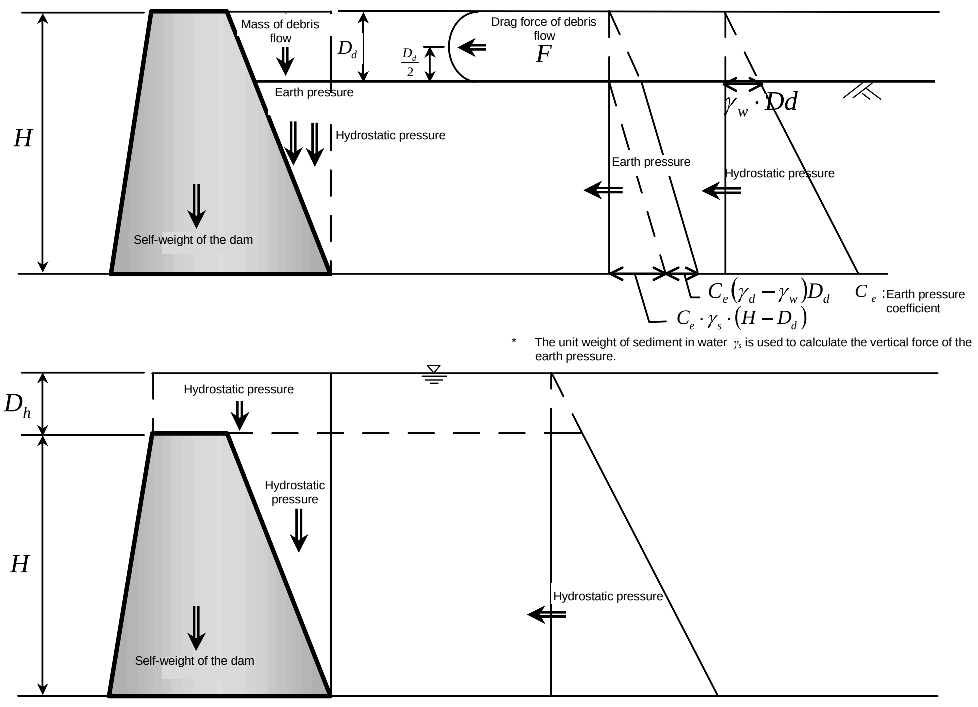


Figure 3 Closed type Sabo dam: design external forces of the overflow section

(H < 15 m, Top: during debris flow, Bottom: during flooding)

(3) Design discharge

> The design discharge consists of the "discharge bulked with sediments" (during flood) and the peak discharge of debris flow (during debris flow). The discharge of water containing fine sediments is calculated by taking the larger value out of the design scale's rainfall of annual exceedance probability and the past maximum rainfall.

Explanation

In principle, the "discharge bulked with sediment" is calculated by the method described in the *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* , Section 2.6.4. For the calculation, the larger value of design scale's rainfall of annual exceedance probability and the past maximum rainfall is taken. In addition, 1.5 times of discharge without sediment is applied.

The debris flow peak discharge is calculated based on the method shown in *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* , Section 2.6.3.

(4) Design water depth

> Design water depth is defined as the overflow depth of the spillway where the design discharge can flow.

Explanation

The design water depth is the largest of the values from the calculation results of 1) to 3).

1) Value of overflow depth in regards to the discharge bulked with sediments

The overflow depth in regards to discharge of water containing fine sediments is calculated using Equation (3) as presented in the *Technical Criteria for River Works*, Design, II, Chapter 3.

$$Q = \frac{2}{15} C \sqrt{2g} \left(3B_1 + 2B_2\right) D_h^{3/2} \quad \text{......(3)}$$

Where,

Q : discharge of water containing fine sediments (m^3/s),

C : coefficient of discharge (0.6 to 0.66),

g : gravitational acceleration (9.81 m/s^2),

B_1 : base width of the spillway (m),

B_2 : width of the overflow (m),

D_h : overflow depth (m),

2) Value of the overflow depth in regards to the debris flow peak discharge.

The overflow depth in regards to the debris flow peak discharge is calculated by the method presented in the *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* , Section 2.6.5 using the design sediment deposition slope.

3) Maximum boulder size

The maximum boulder size is calculated with the method provided in Section 2.6.8 of the *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* .

For the dam in the most downstream of a river that satisfies the debris flow and driftwood sediment and drift wood control plan (100% of control ratio), the design water depth of the spillway should be determined based on the "discharge of water containing fine sediments" (during flood). In such case, the width of the spillway is determined appropriately by considering the current river width and the downstream channel width. However, even in such case, erosion countermeasures in downstream are implemented considering the possibility of flood overflowing the wing.

2.1.3.2 Main body structure

(1) Spillway section

In principle, the spillway section of a Sabo dam is determined by adding the freeboard height to the design water depth. Further, the spillway width shall be set based on the riverbed width before the facilities being built and should be at least 3 m.

Explanation

1) The freeboard is set based on Table 2. However, the freeboard height shall be designed according to the riverbed slope and freeboard height ratio to the design water depth must not be less than the value presented in Table 3. The riverbed slope referred here is the design sediment deposition slope.

Table 2 Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6 m
200 – 500 m^3/s	0.8 m
500 m^3/s or more	1.0 m

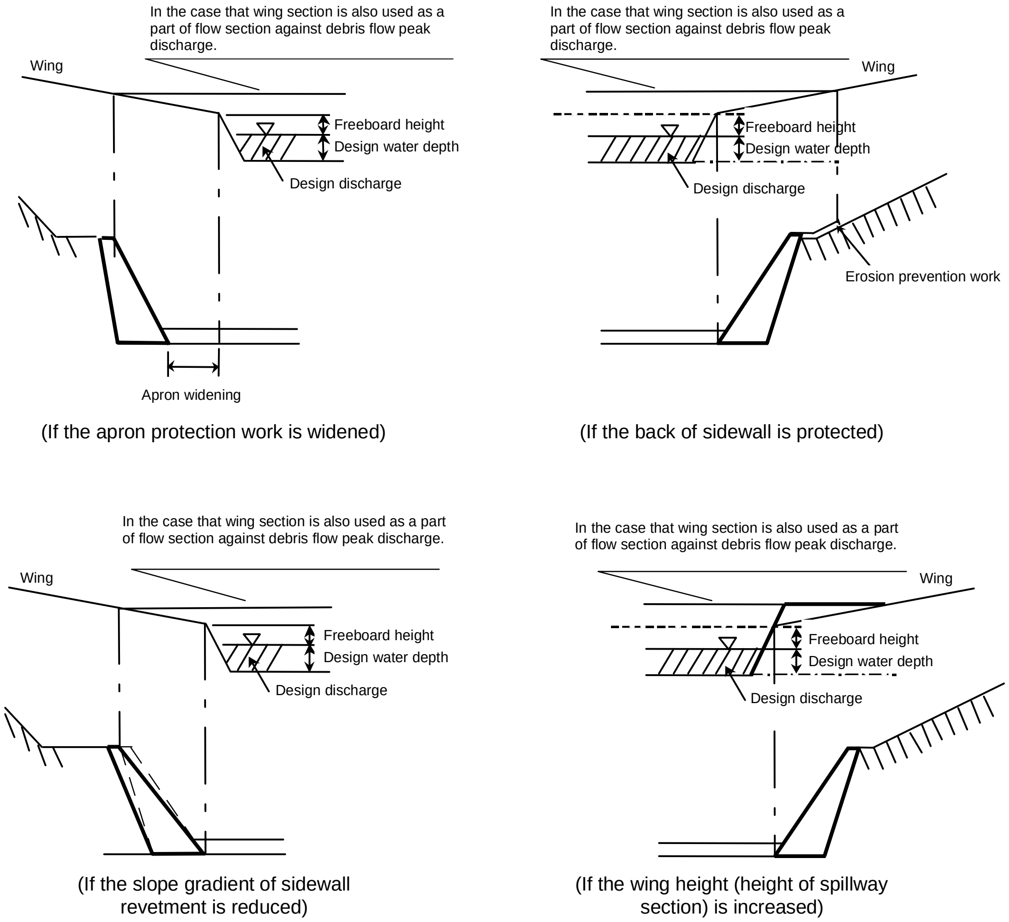
Table 3 Minimum value of the ratio of the freeboard height to the design water depth by riverbed slope

Riverbed slope	(Freeboard height)/(Design water depth)
1/10 or more	0.50
1/10 – 1/30	0.40
1/30 – 1/50	0.30
1/50 – 1/70	0.25

2) When spillway section is designed based on “overflow depth of debris flow peak discharge” or “maximum boulder size”, if sufficient flow area cannot be secured in spillway section due to topography or other factors, wing section can be included as a flow section (see Figure 4). In that case, the design water depth for determining spillway section shall be decided by the overflow depth of discharge of water bulked with sediments.

   In addition, considering the stability of the wing section, damage to apron, and prevention of scouring downstream, appropriate measures shall be taken such as widening of apron, protection of backfill soil of sidewall, reducing the slope of sidewall etc.

   In particular, if properties exist just immediately on the downstream, the above points must be given particular consideration.



(If the apron protection work is widened)

(If the back of sidewall is protected)

(If the slope gradient of sidewall revetment is reduced)

(If the wing height (height of spillway section) is increased)

Figure 4 Spillway section

(Example of treatment when the wing section is also used as a part of flow section against debris flow peak discharge)

(2) Crest width

The crest width of the dam body shall be determined to prevent its failure by collision of boulder and driftwood.

Explanation

The crest of a Sabo dam body must be wide enough to withstand the collision with the volume of conveyed sediment, while the spillway must be wide enough to withstand the abrasion of passing debris. The crest width shall be twice the diameter of the largest debris colliding with the structure, if the body material is mass concrete. However, if the crest width shall be 3 m or more and if the required crest width exceeds 4 m, additional protection using buffer materials (materials which are expected to have buffering effect), embankments, and reinforcement by reinforcing bars or steel frames shall be provided. The effectiveness of the buffering material is confirmed by testing.

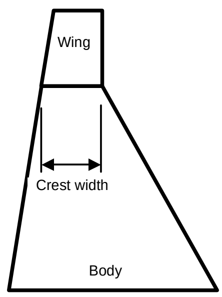


Figure 5 Example of Sabo dam side section and name of the members

(3) Downstream slope

The downstream slope of a Sabo dam shall be highly resistant to damage by overflowing sediment. The gradient of the downstream slope on the overflow section of Sabo dam is generally 1:0.2 (rise : run). This gradient may be lower for a river with small grain diameter, small volume of conveyed sediment during a small- and medium-sized flood, and hillslope area.

Explanation

The downstream slope shall be equal or steeper than the slope calculated by the following formula even if gentle slope is required. U (m/s) means the flow velocity when volume of conveyed sediment starts increasing and H (m) means the Sabo dam height.

$$\frac{L}{H} = \sqrt{\frac{2}{gH}}U \quad \text{.......... (5)}$$

However, its upper limit is 1:1.0.

The flow velocity U (m/s) at which sediment begin to be actively transported shall be about 50% of the flow velocity used by the design external forces (the Guideline, Section 2.1.3.1 (2)). The higher the Sabo dam height, the smaller the value of L / H, but its lower limit is 0.2.

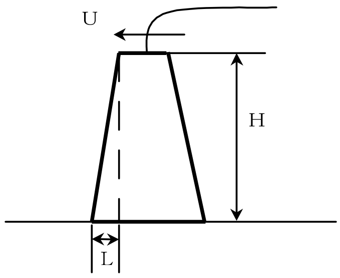


Figure 6 Downstream slope

(4) Ground

The bottom of a Sabo dam should be in contact with rock, but if this is impossible, floating foundation may be used. But in such cases, the dam body height of the Sabo dam shall be less than 15 m.

Explanation

The bottom of a Sabo dam should be in contact with rock to ensure safety. However, if bedrock cannot be found at the Sabo dam location planned based on the debris flow and driftwood countermeasure plan and the debris flow and driftwood countermeasure structure layout plan, floating foundation may be used. In this case, the height of the Sabo dam shall be 15 m or less, in principle.

If the bearing ground is soft or the specified bearing capacity cannot be obtained, ground improvement shall be performed.

(5) Drain hole

The drain hole should be designed to ensure its functions and safety.

Explanation

The drain holes are installed to switch running water during construction, prevent ponding, reduce water pressure after sedimentation, etc. In principle, the drain hole size, shape, quantity, and layout are designed in consideration of sudden discharge of sediment from drain holes, concentration of stress on the drain area, etc.

2.1.3.3 Stability and structure of non-overflow section

(1) Non-overflow section stability calculations

The cross-section of the main body of the non-overflow section shall ensure the same stability as in the overflow section against the design external forces.

Explanation

The cross-section of the main dam body of closed type dam shall be designed uniformly the same for both the overflow and non-overflow sections based on the stability against the design external forces for each section. However, this shall not be applied in special circumstances such as when the ground characteristics of overflow section is different from that of the non-overflow section. In the cross-section where the dam height, *H*, is identical with the overflow section, the stability of non-overflow section is calculated by applying the drag force of debris flow to act horizontally. Such calculation is assuming that sediments are deposited up to the crest of the spillway, including the wings. Stability conditions comply with the Guideline, Section 2.1.3.1(1), while the design external forces comply with the Guideline, Section 2.1.3.1(2), but the acting position complies with Figure 7.

Even though, as described in Explanation 2) of this Guidelines Section 2.1.3.2(1), when debris flow peak discharge is controlled by the cross-section of the spillway and wings, stability should be calculated for multiple cross-sections by assuming the sedimentation surface as in (a) and (b) shown below.

(a) If the depth of debris flow does not exceed the wing height when sedimentation surface is same as crest height of spillway in the cross-section, stability should be calculated with the sediment deposited up to the spillway crest.

(b) If the height of debris flow exceeds the wing height when sedimentation surface is same as crest height of spillway in the cross-section where the stability is calculated,, the sedimentation surface should be lowered so that the debris flow height does not exceed the wing height. The purpose of this arrangement is that the whole drag force of debris flow acts on the dam including wing section.

Furthermore, (i) and (ii) below can be considered as the section where the stability is calculated. However, other positions may be determined considering site conditions, dam size, etc.

(i) Cross-section at the wing edge

(ii) Cross-section where the depth of debris flow matches the wing height

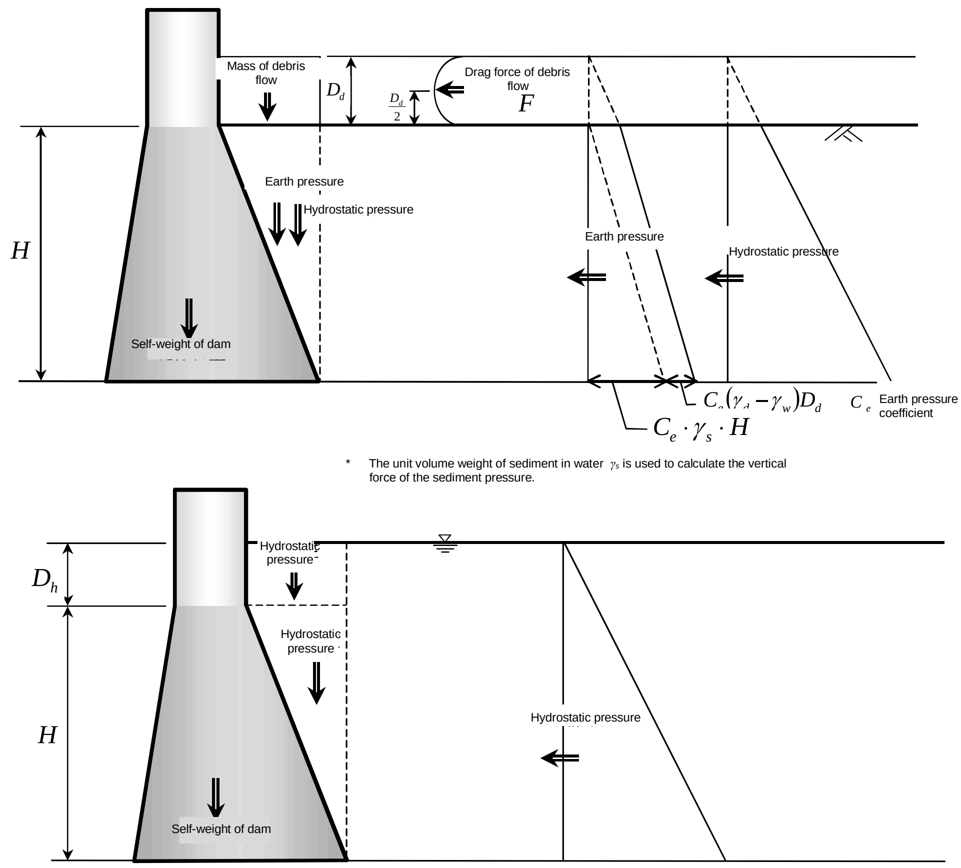


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Figure 7 Closed Type Sabo dam: Design external forces of the Non-overflow section

(H < 15 m, Top: during debris flow, bottom: during flooding)

(2) Structural calculations for wing failure

The structure of Sabo dam wings shall be safe against the drag force of debris flow and the largest value among the impact loads of boulder and driftwood.

Explanation

The cross-section of the wing must satisfy the following four conditions:

a. In principle, the upstream slope of the wing is vertical.
b. The downstream slope of the wing is either vertical or conforms to the downstream slope of the dam body.
c. If the downstream slope of the wing conforms to the downstream slope of the dam body, the lower limit of the crest width of the wing is 1.5 m.
d. The safety factor against shear friction generated by the design external forces at the boundary of main dam body and the wing shall be equal to or more than four.

The design external forces used for the above study are the following three components, and the locations where these act on the wing are as shown in Figure 9.

- Self-weight of the wing
- Drag force of debris flow
- The larger value between impact load of boulder and impact load of driftwood

If the safety factor against shear friction at the boundary of main dam body and wing is equal or less than 4, it should be increased by enlarging the crest width to the upstream side (Figure 8) or equipping the upstream side of the wing with buffer material to reduce the impact load. When protecting the wing with buffer material, the effectiveness of the buffer material should be confirmed by a test.

Since the impact load that causes wing failure[4),5)] is a short-time loading, tensile stress generated at the boundary of main dam body and wing is principally equal or lower than the allowed tensile stress. In contrast, if the tensile stress is higher than the allowed tensile stress, this tensile stress is resolved by reinforcing bars or a steel frame. These reinforcing bars or steel frame are arranged to connect the main dam body and the wing.

In the calculation of the impact loads of boulder and driftwood, the following assumptions are set: their velocity is equal to that of the debris flow, the diameter of boulder is the maximum boulder diameter (D_{95}), the diameter of driftwood is the maximum driftwood diameter (R_{wm}), and the length of the driftwood is the maximum driftwood length (L_{wm}). The boulder and driftwood are assumed to collide with the wing body at the surface of debris flow height with sediment deposition until the crest of spillway (Figure 9 (b)). If the depth of the debris flow is smaller than the boulder and driftwood diameters, it is assumed that the boulder and driftwood flow down and collide with the structure over the sedimentation surface. The flow velocity and depth of a debris flow are calculated by the method described in the *Technical Guideline for Establishing*

Sabo Master Plan against Debris Flows and Driftwood , Section 2.6.5.

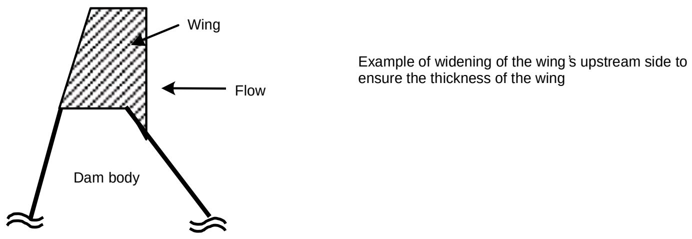


Figure 8 Cross-section of the wing

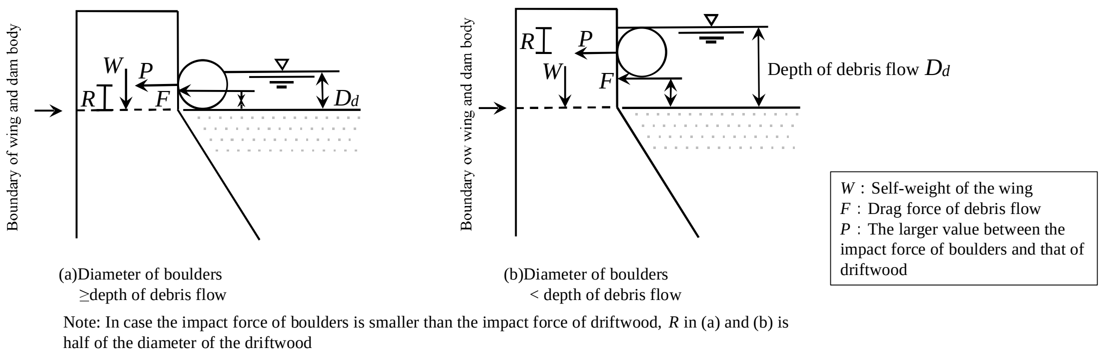


Figure 9 Acting point of design external forces at the boundary between the main dam body and wing

(3) Wing edge

The wing edge of a Sabo dam shall be 1:0.5 or less.

Explanation

The wing edge of the debris flow and driftwood capturing work has a slope of 1:0.5 or less to deal with the failure caused by the impact of a boulder or driftwood.

(4) Slope of the wing crest

In principle, the slope of the wing crest shall be equal to the riverbed slope before the facilities being built (θ_0).

Explanation

In principle, wing crest shall be sloped up to hillslopes. However, if the sloped crest section is too long, it can be shortened to appropriate length according to the site conditions and connected to a horizontal crest section.

2.1.3.4 Downstream riverbed protection work

> Downstream riverbed protection work shall be constructed on the downstream of a Sabo dam as necessary to prevent the main body failure due to scouring.

Explanation

Downstream riverbed protection works are designed using the design discharge (discharge used to determine the spillway section). If the debris flow is expected to overflow the wing, its structure should consider the overflow of a debris flow as shown in Figure 4. The overflow depth of debris flow peak discharge is used for the design of thickness and length of apron.

The downstream slope of the sub-dam conforms to the concept in the Guideline, Section 2.1.3.2 (3).

The spillway section of a sub-dam is basically identical to the spillway section of the main dam. However, when installing a driftwood countermeasure structure on a sub-dam, no freeboard shall be planned. Its structure is determined for the design discharge in accordance with the *Technical Criteria for River Works* (Tentative), Design, Chapter 3.

The design criteria of driftwood countermeasure structure in bedload transport zone (see Appendix D p.66) can be applied for the design of it on the sub-dam.

2.1.4 Structure of open type Sabo dam

2.1.4.1 Stability of the overflow section

> The entire dam body of an open type Sabo dam shall be stable against sliding, overturning, and sinking capacity. Further, permeable section and other parts of the dam body shall be safe against debris flow and driftwood.

Explanation

An open type Sabo dam shall be designed so that the dam body consisting of several types of members shall be integrally resisting the design external forces. In addition, when sediment is used as filling material and water flows constantly (e.g., a large basin), the dam should be designed to secure redundancy. For instance, using soil cement to solidify the sediment filling material, hence partial damage may not expand to the whole body.

(1) Stability conditions

> The stability conditions of the open type Sabo dam are identical to those of a closed type Sabo dam.

Explanation

The concept of the stability conditions of an open type Sabo dam is identical to the closed type Sabo dam (see the Guideline, Section 2.1.3.1 (1)).

(2) Design external forces

> The design external forces of an open type Sabo dam almost complies with the closed type Sabo dam, but the design external forces shall act according to the structure of the permeable section.

Explanation

1) The self-weight is calculated without boulder and water on the permeable section.
2) The earth pressure and hydrodynamic force shown in Figure 10 are treated as external forces to examine the stability of the entire dam body and the safety of each member. Since the weight of debris flow can act as a surcharge pressure, the earth pressure is distributed in a trapezoid shape.

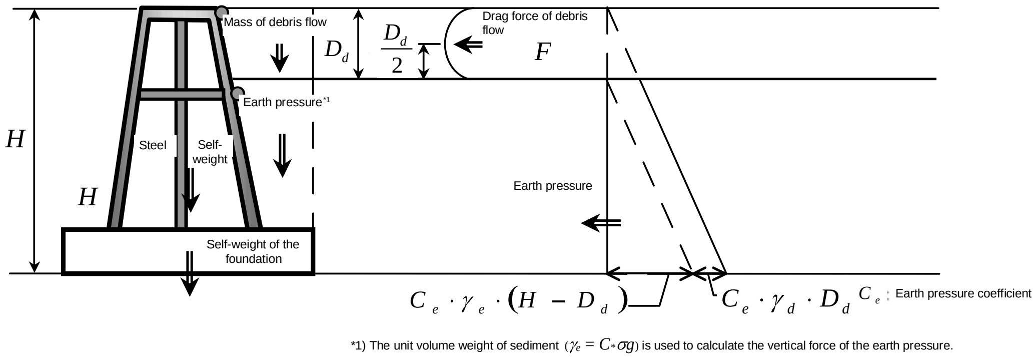


Figure 10 Design external forces (during debris flow)

3) If the permeable part is made of concrete, the self-weight of the dam body is calculated using the dam body block volume (V_c) which is calculated by assuming the overflow section as a closed structure, and the dam body block weight (W_{rc}) which is calculated by assuming the overflow section as an open structure (Figure 11). Further, it is noted that the dam body block refers to the concrete block in the permeable part, not the concrete block used at the time of casting.

$$\gamma_{rc} = W_{rc} / V_C \quad \text{...} (6)$$

Where,

γ_{rc} : apparent unit weight of concrete (kN/m^3),

W_{rc} : dam body block weight (kN) calculated by assuming the overflow section as an open structure with concrete,

V_c : dam body block volume (m^3) calculated by assuming the overflow section as a closed structure.

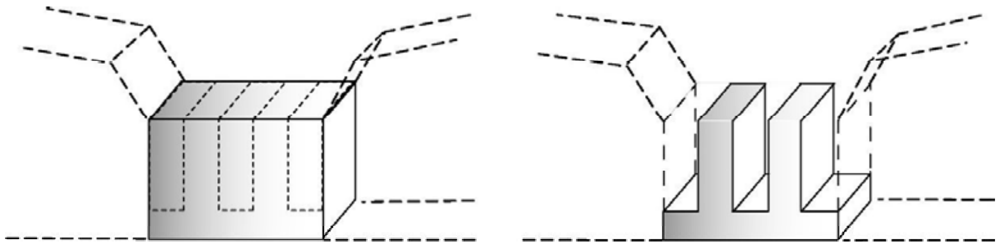


Figure 11 Dam body volume of spillway of the slit part

4) Combinations of design external forces excluding the self-weight of dam body are as shown in Table 4.

Table 4 Design external forces used to calculate stability of an open type Sabo dam (excluding self-weight)

	Normal time	During debris flow	During flood
Dam height less than 15 m		Drag force of debris flow, Earth pressure	
Dam height of 15 m or more		Drag force of debris flow, Earth pressure	

The stability conditions for the permeable section of an open type Sabo dam with height of 15 m or more are identical to those with height of less than 15 m. Besides, since the upstream slope of the non-overflow section is generally steep, the safety factor should be examined in a state where seismic inertia force acts from the downstream side when it is not fully filled with sediment.

(3) Design discharge

> The design discharge shall be the debris flow peak discharge used to design the spillway section.

Explanation

The debris flow peak discharge is calculated based on the method in the *Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* , Section 2.6.3.

(4) Design water depth

> The overflow depth of the spillway where the design discharge pass through is defined as the design water depth.

Explanation

Design water depth is the largest of the values of 1) and 2) below. However, if enough flow area cannot be secured in spillway section due to topography or other factors, wing section can be included as flow section.

1) Value of the overflow depth for debris flow peak discharge
   (see the Guideline, Section 2.1.3.1(4))
2) Value of the maximum boulder size
   (see the Guideline, Section 2.1.3.1(4))

For a dam located in the most downstream of a river that satisfies the debris flow and driftwood control plan (with 100% control ratio), the design water depth of the spillway, as in the case of a closed type Sabo dam, should be determined based on the Explanation of Section 2.1.3.1(4) of this Guidelines. However, if the “debris flow peak discharge” is smaller than the “discharge bulking with sediment” (during flood), the design water depth of spillway shall be determined based on the “debris flow peak discharge”.

2.1.4.2 Structural design of the permeable section

(1) Structural design conditions

Members of permeable sections shall be safe against the design external forces. The Sabo dam structure shall be as redundant as possible to achieve fail-safe performance, hence the failure of one member will not cause the failure of entire dam.

Explanation

The safety of the strength of members of the permeable section must be confirmed. To deal with sediment movement phenomena, the structure must have high redundancy so the failure of some members will not cause the failure of the overall structure. This is because sediment movement such as debris flows can cause severe damage, requiring countermeasures that take into account the uncertainty of their scale and the possibility of temporally and spatially complex phenomena. Because sediment movement phenomena such as debris flow has many uncertain elements yet causes severe damage.

The following items must be the part of the structural examination.

1) Examination of the strength of each member against drag force of debris flow and earth pressure
2) Examination of the strength of each member against thermal stress caused by temperature change
3) Examination of the strength of joints against the forces in 1) and 2) above
4) Examination of strength of each member against the impact load of boulder and driftwood.

For the members installed to capture debris flows (functional members) but not to maintain the shape of structure (structural members), plastic deformation is allowed provided that the boulder of debris flow can be captured.

Note that the following points should be additionally considered for a site where the external forces in the target basin are severe.

- In a site with particularly severe external forces, site conditions and target basin characteristics shall be thoroughly investigated and the boulder size should be determined appropriately. If record of volume of conveyed sediment in nearby rivers is available, the boulder size in the record can be used as a reference.
- If extremely large boulder could flow down with particularly severe external force, a dam structure shall be designed to maintain its capturing function as a whole Sabo dam even if such large boulders collide with the dam.

(2) Design external force

Design external forces considered in the structural design are the self-weight, drag force of debris flow, earth pressure, and thermal stress.

Explanation

Combinations of design external forces considered in the structural design are shown in Table 5.

Since the design external forces are short-time loading during debris flow, the allowable stress shall be increased by 1.5 times by considering the past records whereas since the earth pressure acts for a long period after the debris flow is captured, the allowable stress when the Sabo dam got fully filled with sediment is not increased. For the thermal stress, the allowable stress is generally increased by 1.15 times. If the thermal stress is large, the cross-section of the member shall be designed so that it is not determined by the thermal stress, or the length of the member shall be divided.

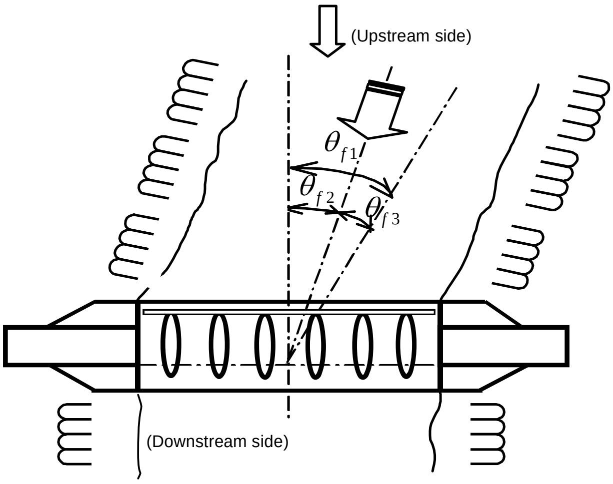
In structural calculations for an open type Sabo dam, the generated stress of members and the strength of joints must be safe against combination of design external forces during debris flow and when the structure is full with sediment. Further, if the facility is a statically indeterminate structure, its safety against combinations of design external forces during temperature change must be confirmed.

Permeable sections are designed so that they are safe from eccentric load, in addition to the loads in Table 4. Such eccentric loads are the drag force of debris flow which acts eccentrically on a structure and the impact load generated by the collision of boulder and driftwood are also considered.

Furthermore, the axis of a Sabo dam on a curve section of a river should be roughly at right angles to the downstream river channel. However, consideration must be given to minimize its eccentricity to the upstream river course to maintain its capturing functions. If the dam is eccentric to the upstream stream axis, the eccentric angle to the Sabo dam (θ_{f1}) is set by hypothesized debris flow longitudinal axis to the dam axis (θ_{f2}) with the allowed angle (θ_{f3}) (See Figure 12). Additionally, when the dam is constructed on a curve section, the risk that the inner side of the curve section may not be blocked by boulders contained in the front part of the debris flow and subsequent flows to pass through must be addressed.

Table 5 Combination of Design external forces Considered in the Structural Design

Cases	During debris flow	When filled with sediment	During temperature change
Self-weight	✓	✓	✓
Drag force of debris flow	✓		
Earth pressure	✓	✓	
Thermal stress			✓
Allowable stress increase factor	1.5	1	1.15



$$\theta_{f1} = \theta_{f2} + \theta_{f3} \quad \cdots (7)$$

θ_{f1} : eccentric angle to the sabo dam (°)

θ_{f2} : angle of the hypothetical longitudinal axis of the debris flow and the axis perpendicular to the dam axis (°)

θ_{f3} : allowable angle (θ_{f3} = 10°)

Figure 12 Eccentric Loads on Permeable Section
(In case Sabo dam installed on a Curve Section of a River)

2.1.4.3 Main body structure

(1) Spillway section

The spillway section shall be identical to that of a closed type Sabo dam, but it shall be a section that allows the debris flow peak discharge to pass safely even after the permeable section (slit part) is blocked.

Explanation

The spillway shall have a sufficient cross-section to allow the debris flow peak discharge to pass even after the permeable section has been completely blocked by debris. The freeboard does not need to be considered in this case.

However, if enough flow area cannot be secured in spillway section due to topography or other factors, wing section can be included as flow section.

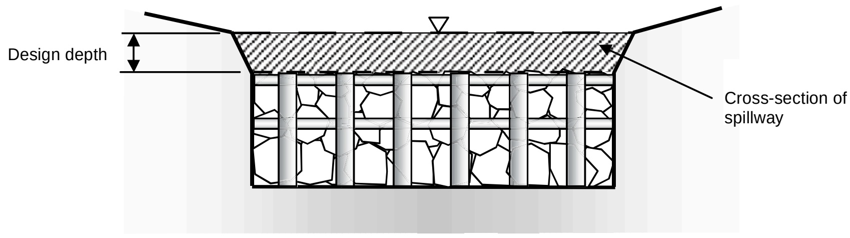


Figure 13 Cross-section of a spillway (the part in diagonal pattern)

(2) Setting of the opening section

The width, height, and layout of the opening section in an open type Sabo dam shall be determined so that it can effectively capture debris flow and driftwood.

Explanation

The opening section must be wide enough so that the permeable function of an open type Sabo dam is fully achieved.

The height of opening section shall be equal to or greater than the depth of debris flow or of flood discharge bulked with sediment used to obtain the design capturing volume of debris flow and driftwood.

The bottom surface of the opening section shall be designed so that the discharge at normal times can flow downstream without damming up when the structure is not fully filled with sediment.

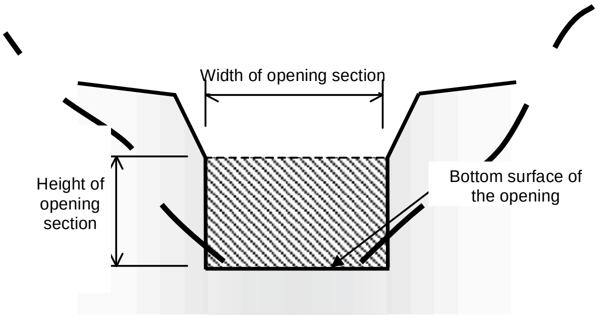


Figure 14 Opening section of an open type Sabo dam (the part in diagonal pattern)

(3) Setting the cross-section of the permeable section

The cross-section of the permeable section of an open type Sabo dam shall be determined based on the maximum boulder size and the purpose of the structure.

Explanation

By optimal setting of spacing of the permeable section, an open type Sabo dam for debris flow capturing can have debris flow capturing function while allowing sediment flow at normal time (see Figure 15). Thus, the flow type of a debris flow, maximum boulder size (D_{95}), layout of existing structures in the target basin, and Sabo dam height shall be fully considered when designing the cross-section of the permeable section.

The horizontal span is set to about 1.0 times of the maximum boulder size (D_{95}). When the height of an open type Sabo dam is planned to be higher than the debris flow depth, the vertical length of each mesh opening is set to 1.0 times of the maximum boulder size (D_{95}) to ensure the debris flow capture. The height of the lowest permeable section (see Figure 15) shall be less than the depth of debris flow but it shall be larger than the heights of upper permeable sections (see Table 6).

According to various experiments (see Figure 16), if the horizontal and vertical length of each mesh opening are smaller than 1.5 times the maximum boulder size (D_{95}), the overflow section may be blocked when the volumetric sediment concentration is high. Therefore, the horizontal and vertical length of each mesh opening can be widened to 1.5 times in the necessary cases. One of the necessary cases is, for example, that when several open type Sabo dams are planned, the design sediment and driftwood discharge volume could be efficiently controlled by designing the upper dam with widened spacing.

If all the following conditions are met as far as sediment discharge at normal time would pass through, permeable section can be designed by a method other than above-mentioned method with considering the conditions of river.

1) The permeable section lower than the debris flow depth is surely blocked by boulders contained in the debris flow and the blockage remains during debris flow.
2) The permeable sections higher than the debris flow depth are surely blocked by the following part of debris flow and the blockage remains during the latter half of debris flow.

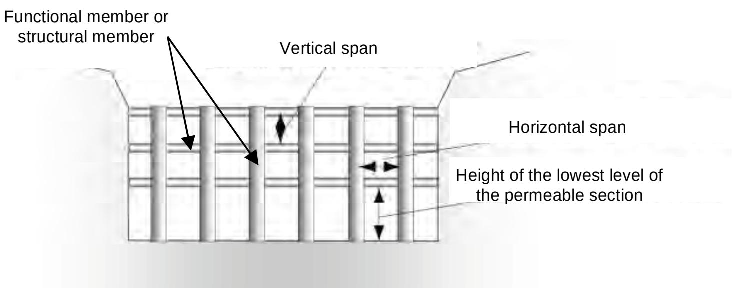


Figure 15 Span of the permeable section

Table 6 Setting of the permeable section of an open type Sabo dam

Function	Horizontal span	Vertical span	Height of the lowest level of permeable section
Capturing debris flows	$D_{95} \times 1.0$ *1	$D_{95} \times 1.0$ *1	Lower than debris flow depth *2

*1 As stated above, the horizontal and vertical span can be widened to 1.5 times the maximum boulder size (D_{95}).

*2 As stated above, it should be noted that the height of the lowest level shall not be smaller than the vertical length of each mesh span of other level

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(Appendix B) Blockage of the permeable section (experiment results)

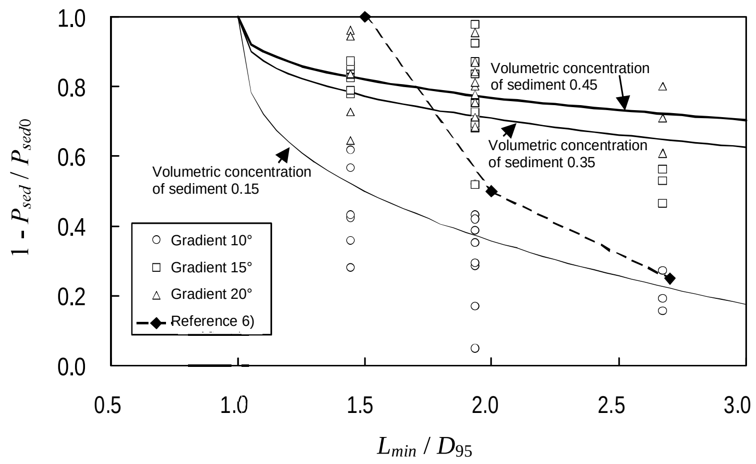


Figure 16 Relationship between the width of the mesh opening (steel pipe spacing) and the decreasing rate of the peak sediment load

Relationship between the width of the mesh opening (steel pipe spacing) and the decreasing rate of the peak sediment load. P_{sed}: peak sediment load at the downstream end of a channel with a structure, P_{sed0}: peak sediment load at the downstream end of a channel without a structure, L_{min}: smaller one[7] of horizontal span and vertical span in a grid type Sabo dam, the horizontal span[6] in another Sabo dam without horizontal beams, D_{95}: grain size for which 95% of a material weight is finer[7], but maximum grain size[6]. As the volumetric concentration of sediment in a debris flow decreases, $1 - P_{sed} / P_{sed0}$ also decreases, hence the permeable section is not easily blocked.

～～～

2.1.4.4 Stability and structure of the non-overflow section

The cross-section of the body of the non-overflow section shall be decided based on the stability calculation.

Explanation

The concepts of stability conditions of the non-overflow section of an open type Sabo dam and design external forces used are identical to those of closed type Sabo dam (this Guidelines, Section 2.1.3.3).

2.1.4.5 Downstream riverbed protection work

Downstream riverbed protection works shall be designed as needed by considering the local geology, topography, and other factors to maintain the stability of the Sabo dam body.

Explanation

In an open type Sabo dam, the water in normal time continues to flow on the riverbed almost identically prior to the dam construction. Therefore, downstream riverbed protection works are often considered to be unnecessary. However, if scouring is predicted to occur due to subsequent flow of the captured debris flow or relatively large gap between the level of the surface of the foundation of the permeable part and that the level of the downstream riverbed exists, downstream riverbed protection works identical to those for a closed type Sabo dam are required. The necessity of water cushion or a sub-dam is carefully examined and planned.

Note that the cross-section of the spillway of a sub-dam is designed by adding freeboard to the cross-section of the spillway of the main dam set in the Guideline, Part 2.1.4.3 (1).

2.1.5 Structure of a semi-open type Sabo dam

2.1.5.1 Stability of the overflow section

The entire dam body of a semi-open type Sabo dam shall be stable against sliding, overturning, and bearing capacity. The permeable section and other members of the dam body shall be safe against debris flow and driftwood.

Explanation

A semi-open type Sabo dam shall be designed so that the dam body consisting of several types of members shall be integrally resisting the design external forces. In addition, when sediment is used as filling material and water flows constantly (e.g., a large basin), the dam should be designed to secure redundancy. For instance, using soil cement to solidify the sediment filling material, hence partial damage may not expand to the whole body.

(1) Stability conditions

The stability conditions of the semi-open type Sabo dam is identical to those of a closed type Sabo dam.

Explanation

The stability conditions of a semi-open type Sabo dam shall conform to the closed type Sabo dam (see the Guideline, Part 2.1.3.1(1).)

(2) Design eternal forces

The design external forces of a semi-open type Sabo dam is basically identical to those of a closed type Sabo dam, but the design external forces which applied for open type Sabo dam is also applied according to the design of open part.

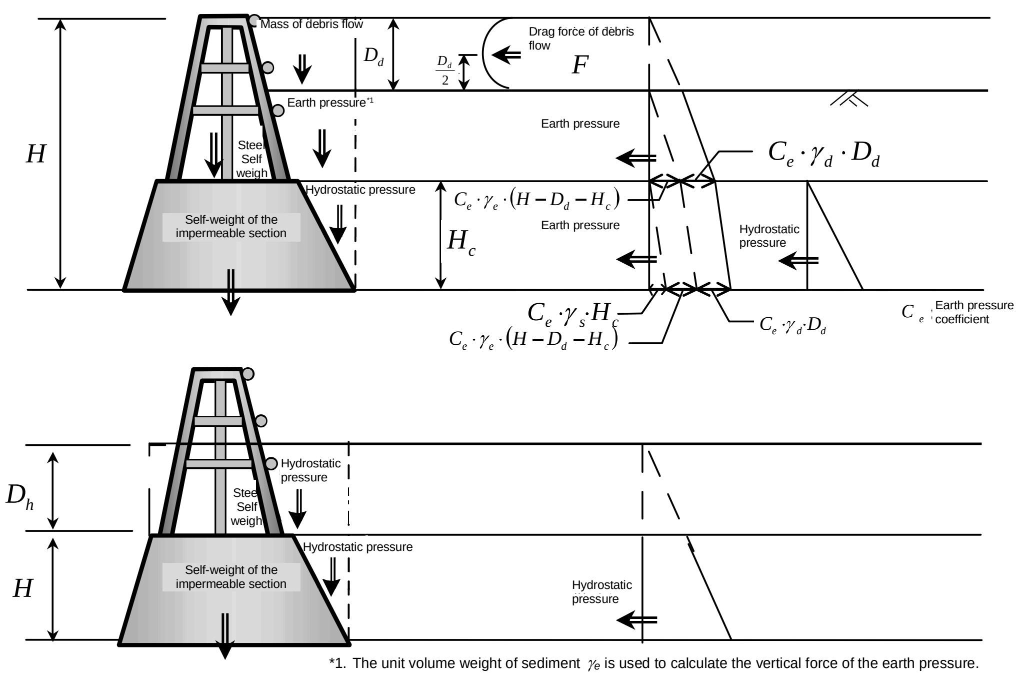
Explanation

1) Table 7 shows the combinations of design external forces used for the stability calculations.

Table 7 Design external forces used for stability calculation of semi-open type Sabo dam (excluding self-weight)

	Normal time	During debris flow	During flood
Dam height less than 15 m		Hydrostatic pressure, earth pressure, drag force of debris flow	Hydrostatic pressure
Dam height of 15 m or more	Hydrostatic pressure, earth pressure, uplift pressure, inertia force during an earthquake, hydrodynamic pressure during an earthquake	Hydrostatic pressure, earth pressure, uplift pressure, drag force of debris flow	Hydrostatic pressure, earth pressure, uplift pressure

2) Design external forces used for stability calculation act both on the permeable section and on the impermeable section as shown in Figure 17.



*1. The unit volume weight of sediment γ_e is used to calculate the vertical force of the earth pressure.
*2. The unit volume weight of sediment in water γ_s is used to calculate the vertical force of the earth pressure.

Figure 17 Design external forces used for stability calculations of a semi-open Sabo dam (H < 15m, Top: during debris flow, bottom: during flood)

3) Self-weight of the permeable section is calculated by assuming that boulder and water do not block the permeable section. Further, the self-weight of water that passes through the permeable section during a flood acts on the impermeable section as hydrostatic pressure.

(3) Design discharge

The design discharge shall be identical to that of closed type Sabo dam.

Explanation

The concepts of design discharge of a semi-open Sabo dam are identical to that of closed type Sabo dam (see the Guideline, Section 2.1.3.1 (3)).

(4) Design water depth

The design water depth shall be identical to that of a closed type Sabo dam.

Explanation

The concept of the design water depth of a semi-open Sabo dam is identical to that of a closed type Sabo dam (see the Guideline, Section 2.1.3.1 (4)).

2.1.5.2 Structural examination of the permeable section

The examination of the structure of the permeable section shall be conducted in the same method as that of an open type Sabo dam.

Explanation

The members and structure of a semi-open Sabo dam are examined in the same method as those of an open type Sabo dam (see the Guideline, Section 2.1.4.2).

2.1.5.3 Main body structure

(1) Spillway section

The spillway section shall be identical to that of an open type Sabo dam.

Explanation

The spillway section of a semi-open Sabo dam is identical to that of an open type Sabo dam (see the Guideline, Part 2.1.4.3(1)).

(2) Setting the opening section

The opening section shall be determined in the same method as that of open type Sabo dam.

Explanation

The opening section of a semi-open Sabo dam is determined in the same method as that of open type Sabo dam (see the Guideline, Section 2.1.4.3 (2)).

(3) Setting the permeable section

The permeable section shall be set in the same method as that of a open type Sabo dam.

Explanation

The permeable section of a semi-open type Sabo dam is designed in the same method as that of a open type Sabo dam (see the Guideline, Part 2.1.4.3 (3)).

(4) Crest width of the impermeable section

The crest width of the impermeable section shall be determined so that it does not fail against the impact load of boulder and driftwood.

Explanation

The crest width of the impermeable section is at least twice the maximum diameter of the colliding boulder (D_{95}). However, considering the safety of the impermeable section according to the closed-type Sabo dam, the crest width shall be at least 3 m.

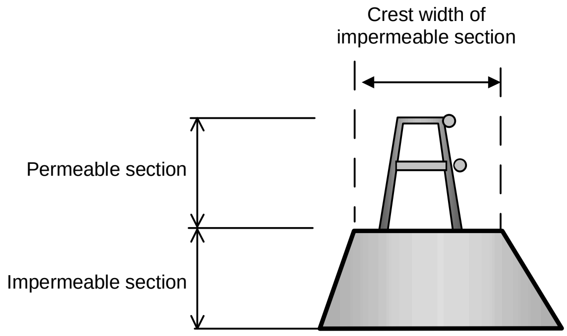


Figure 18 Lateral cross-section of the overflow section of a semi-open Sabo dam (example)

(5) Downstream slope

The downstream slope shall be identical to that of a closed-type Sabo dam.

Explanation

The downstream slope of a semi-open Sabo dam is identical to that of a closed-type Sabo dam (see the Guideline, Section 2.1.3.2 (3)).

(6) Ground

The ground shall be identical to that of a closed type Sabo dam.

Explanation

The ground of a semi-open Sabo dam is identical to that of a closed type Sabo dam (see the Guideline, Section 2.1.3.2 (4)).

(7) Drain hole

The drain hole shall be identical to that of a closed type Sabo dam.

Explanation

The drain hole of a semi-open Sabo dam is identical to that of a closed type Sabo dam (see the Guideline, Section 2.1.3.2 (5)).

2.1.5.4 Stability and structure of the non-overflow section

The stability and structure of the non-overflow section shall be identical to those of a closed type Sabo dam.

Explanation

The stability and structure of the non-overflow section of a semi-open Sabo dam are identical to those of a closed type Sabo dam (see the Guideline, Section 2.1.3.3).

2.1.5.5 Downstream riverbed protection works

> The downstream riverbed protection works of a semi-open Sabo dam shall be identical to that of a closed type Sabo dam.

Explanation

The downstream riverbed protection works of a semi-open Sabo dam is identical to that of a closed type Sabo dam (see the Guideline, Section 2.1.3.4).

Length and thickness of the apron shall be designed considering the more severe conditions between the two: scouring due to flooding and scouring due to the following part of debris flow. The height from the apron crest to the crest of impermeable part is used in the design of “during flood” while the height from the apron crest to the crest of permeable part is used in the design of “during debris flow”.

The necessity for water cushion or a sub-dam should be carefully examined and planned. Note that the cross-section of the spillway of a sub-dam is designed by adding freeboard to the cross-section of the spillway of the main dam set in the Guideline, Part 2.1.5.3 (1).

2.1.6 Sediment and driftwood removal

If the effect of debris flow and driftwood capturing work is estimated by assuming the sediment and driftwood removal, sediment and driftwood removal of captured or deposited sediment and driftwood shall be immediately conducted.

Explanation

Basic concepts of sediment and driftwood removal are based on the Guideline, Section 3.

2.2 Debris flow and driftwood restraint work

2.2.1 Debris flow and driftwood restraint hillside works

Debris flow and driftwood restraint hillside works aim to stabilize hillside slopes by reforestation or other civil engineering structure.

Explanation

Debris flow and driftwood restraint works are mainly hillside conservation works to prevent hillside failure that may trigger debris flows.

2.2.2 Riverbed sediment stabilization work

Riverbed sediment restraint work is a method to prevent the movement of sediment deposited on riverbed such as by groundsill

Explanation

Riverbed sediment restraint work is a method to prevent the movement of sediment deposited on riverbed or riverbanks, mainly by groundsill. In principle, the upstream side of groundsill shall be filled with sediment up to its crest and the structure shall not be directly exposed to the impact load of boulder and driftwood. Further, measures such as sediment accumulation on the upstream side of the wings shall be taken to prevent wing failure due to debris flow. By referring to the Guideline, Section 2.1.3.1(2), the design external forces for this stabilization work is only the hydrostatic pressure, while the load of debris flow is not considered.

The spillway design of groundsill, as the riverbed sediment restraint work, shall follow this Guidelines, Section 2.1.3.2(1). However, the width of spillway shall be as wide as possible considering the topography of site, while if the design water depth is set depending on the debris flow peak discharge, freeboard does not need be considered in the design. Further, for other structures made of concrete, the design must comply with the structure of the closed type Sabo dam as explained in the Guideline, Section 2.1.3.

2.3 Debris flow torrent training work

2.3.1 Cross-section

The cross-section of debris flow torrent training work shall be determined by considering the debris flow discharge and depth, and then a freeboard shall be added. It is noted that the riverbed aggradation in upstream direction shall not cause flooding.

Explanation

Debris flow torrent training works shall be planned after one or more debris flow and driftwood capturing works, such as Sabo dams or debris flow depositing areas in the upstream and be designed to connect to the lower side of them in order to control the debris flow movement to a safe place.

The design discharge is determined by assuming that the debris flow peak discharge decreases by a ratio of the total sediment volume controlled by facilities in the upstream to the design volume of debris flow material. However, it shall not be less than the discharge without sediment (obtained from probable design rainfall) plus 10% of sediment content.

The width of a debris flow torrent training work is at least twice the maximum boulder size of debris flow (D_{95}), or at least 3 m.

If the ratio of the total sediment volume controlled by facilities to the design volume of debris flow material is equal to 1.0, channel work shall be planned instead of debris flow torrent training work. Explanation on the normal channel work is available in Section 3-2 of the Facility Planning Part, *Technical Criteria for River Works*.

The freeboard shall be as follows.

Discharge	Freeboard (ΔD_d)
< 200 m^3/s	0.6 m
200 – 500 m^3/s	0.8 m
500 – 2000 m^3/s	1.0 m

But, it shall not be less than the following values according to the riverbed slope.

Riverbed Slope	ΔD_d / D_d
> 1/10	0.5
1/10 – 1/30	0.4

Where, D_d : water depth (m).

2.3.2 Alignment

The alignment of a debris flow torrent training work shall be as straight as possible.

Explanation

Due to the large inertia of debris flow, the alignment of a debris flow torrent training work should also be straight. If the alignment has to bend due to the topography, land use, or other reason, a circular curve should be inserted. The radius of curvature is obtained by the following equation where its center angle shall be 30° or less[8].

$$\frac{B_r}{R_{(IN)}} \leq 0.1 \qquad (8)$$

Where, B_r: channel width (m), and $R_{(IN)}$: radius of curvature of the bend (m), as shown in Figure 19.

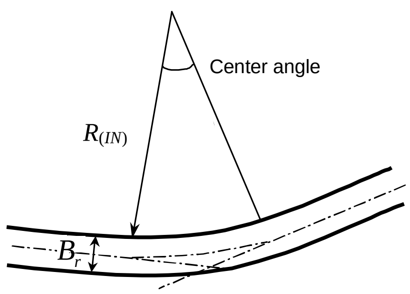


Figure 19 Alignment of a curved debris flow torrent training work

2.3.3 Longitudinal profile

The longitudinal profile of a debris flow torrent training work shall avoid abrupt slope change. If riverbed aggradation in upstream direction is predicted to occur, the debris flow torrent training work structure shall be designed to be safe from it.

Explanation

A debris flow torrent training work shall guide debris flow to a safe place, hence sediment deposition due to abrupt slope change shall be avoided. In addition, if riverbed aggradation in upstream direction is predicted to occur at the end of the river, the debris flow torrent training work structure shall be secured for instance by setting the height of embankment according to the aggradation in upstream direction.

2.3.4 Structure

2.3.4.1 Riverbed

In principle, the debris flow torrent training work shall be an excavated channel.

Explanation

In principle, the debris flow torrent training work shall be an excavated channel to ensure safety.

2.3.4.2 Curved section

On the curved section, the height of the embankment is determined by considering the water level rise on the outer side bank.

Explanation

Water level rise on the outer side bank is estimated based on theoretical values, measured values, and experimental results. The structure of debris flow torrent training work shall allow this water level to pass safely.

In a debris flow, the maximum water level on the outer side bank $D_{d(OUT)max}$ can be as high as $D_{ds} + 10 \cdot \frac{(B_r \cdot U^2)}{(\theta_r \cdot g)}$. However, on an alluvial fan where debris flow torrent training work and channel work are usually constructed, the maximum water level of debris flow and flood are obtained by the following equations respectively [8)].

Debris flow: $$D_{d(OUT)\,max} = D_{ds} + 2\frac{B_r \cdot U^2}{R \cdot g} \quad \text{(9)}$$

Flood (supercritical flow): $$D_{d(OUT)\,max} = D_{ds} + \frac{B_r \cdot U^2}{R \cdot g} \quad \text{(10)}$$

Where:

D_{ds} : depth in the straight section (m),

B_r : channel width (m),

U : average flow velocity (m/s),

R : radius of curvature at the center of the channel (m), and

g : gravitational acceleration (9.81 m/s^2).

2.4 Debris flow depositing area

2.4.1 Debris flow dispersion/depositing area

2.4.1.1 Type

The shape of a debris dispersion/depositing area shall be appropriate based on the flow characteristics of debris flow and topography.

Explanation

The shape of a dispersion/depositing area is determined based on the scale, flow, and inundation characteristics of the past debris flows or if no data of them is available, based on the characteristics of debris flows in similar rivers.

2.4.1.2 Design sedimentation slope

The design sedimentation slope of a debris flow dispersion/depositing area shall be between 1/2 and 2/3 of the original riverbed slope before the facilities being built.

Explanation

The default of the design sedimentation slope of a debris flow dispersion/depositing area is between 1/2 and 2/3 of the riverbed slope before the facilities being built. If an applicable value measured in the past is available, the measured value may be used instead of the default value.

2.4.1.3 Design sediment depositing volume

The design sediment depositing volume of a debris flow dispersion/depositing area is calculated in the condition of sedimentation occurred in design sedimentation slope.

Explanation

The design sediment depositing volume of a debris flow dispersion/depositing area is calculated for a situation where sediment has been deposited on the design sedimentation slope determined by the Guideline, Part 2.4.1.2.

2.4.1.4 Structure

Sabo dams or groundsills shall be constructed at the upstream and downstream ends of a debris flow dispersion/depositing area and revetment work for embankment or groundsill shall be constructed inside the depositing area as necessary.

Explanation

A debris flow dispersion/distribution work consists of Sabo dam (groundsill) at the upstream and downstream ends, dispersion part, deposition part, and downstream end training work. To make gentle slope at the widened part (Figure 20), the part shall be excavated in general. Thus, the upstream end Sabo dam (groundsill) is installed to maintain the gap between the current riverbed upstream and the widened part. The downstream end Sabo dam (groundsill) controls the dispersed flow to smoothly return it to the river course. In some cases, groundsill is constructed in the deposition part to increase its sedimentation capacity.

The width of the deposition part (B_{d2}) should be within about 5 times the width of the upstream channel (B_{d1}).

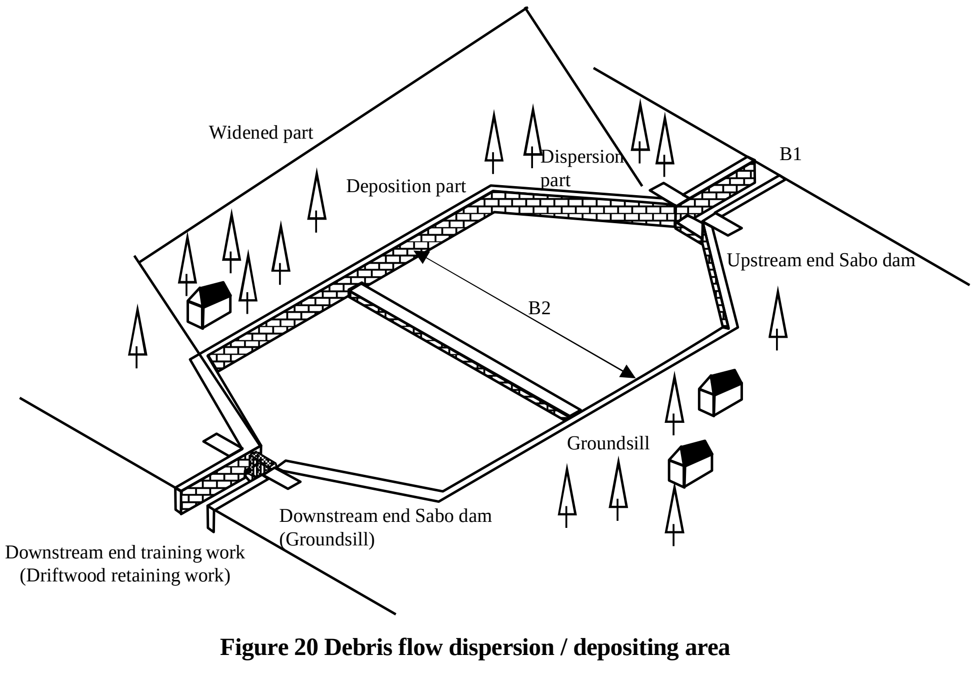


Figure 20 Debris flow dispersion / depositing area

2.4.2 Debris flow depositing channel

> Debris flow depositing channel shall actively deposit sediment of debris flow in a channel on an alluvial fan. Further, revetment work for embankment shall be used to prevent riverbank erosion.

Explanation

To actively deposit sediment of debris flow in the channel, the sediment transport capacity is reduced by having the gentle channel bed gradient and widening the channel width. However, if sediment is deposited at the normal discharge prior to debris flow occurrence, the deposition capacity at the time of debris flow will decrease. Thus, assuming the volume of conveyed sediment at normal time (sediment concentration), the channel bed gradient shall be designed so that sedimentation by the normal discharge does not occur.

2.4.3 Sediment and driftwood removal

> If sediment due to debris flow has been deposited in a debris flow depositing works, this debris shall be removed immediately.

Explanation

The basic concept of debris removal is explained in the Guideline, Part 3.

2.5 Erosion control greenbelt

> An erosion control greenbelt shall be installed near the downstream end of the debris flow deposition zone to lower the velocity of the debris flow in the deposition zone.
>
> The deposition zone consists of a groundsill at the lower end, low flow channel, debris flow direction-controlling works, trees and supplementary facilities. These facilities shall be constructed by considering current topography.

Explanation

(1) Tree species used

Tree species to be introduced are selected with reference to the indigenous tree species existing in the planned area or in nearby locations with similar conditions.

(2) Density of trees

1) The density of trees is determined by ensuring a minimum interval between trees which is necessary for the growth of the trees, reduction of the flow velocity in the forest area, and obtaining an adequate sediment deposition effect.

2) Trees which does not toppled by hydrodynamic force are selected.

(3) Evaluation of effectiveness

The effect of greenbelt consists of the sediment deposition volume obtained by bed load calculation assuming the roughness coefficient increased by the greenbelt, and the volume of entrainable channel deposits in the planned zone which shall not be eroded out due to the greenbelt.

The default value of the planned average deposition depth is approximately 0.3 to 0.5m [9].

(4) Maintenance of the greenbelt

To maintain the forest function as erosion control greenbelt, tree replantation or weeding shall be conducted as necessary.

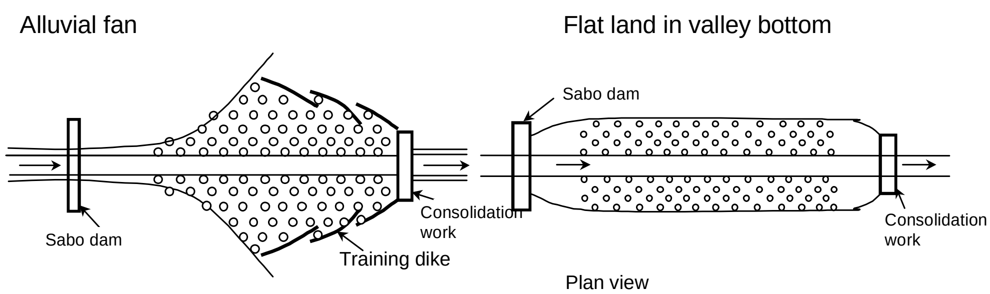


Figure 21 Erosion control greenbelt

2.6 Debris flow direction-controlling work

A debris flow direction-controlling work controls the flow direction of a debris flow. The structure shall be high enough to prevent overflow and scouring at the front slope toe.

Explanation

(1) Alignment of a debris flow direction-controlling work

If a safe location to flow sediment downstream from the reference point is available, and the flowing process would be safe and would not cause any damage to the downstream, the direction of the debris flow can be controlled by debris flow direction-controlling works. To prevent the overflow due to direct impact of debris flow, the alignment of the debris flow direction-controlling work shall be angled to less than 45° (θ_c < 45°). When changing the direction of debris flow by 45 degree or more, more than one debris flow direction-controlling work, open levees in the echelon structure, shall be allocated to change the direction gradually.

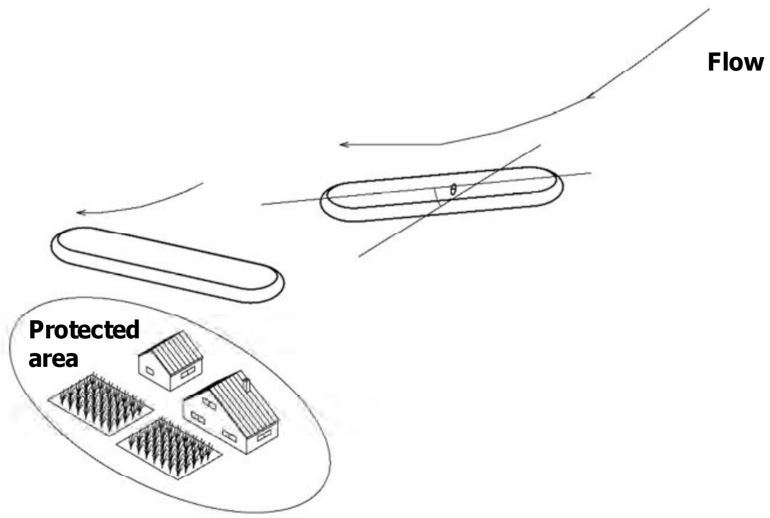


Figure 22 Alignment of a debris flow direction-controlling work

(2) Height of debris flow direction-controlling work

The gradient of the crest of the debris flow direction-controlling work shall be parallel to the current riverbed gradient. The height is the total of the depth of the debris flow and the freeboard (see the Guideline, Part 2.3.1.).

The debris flow velocity and depth are obtained based on the *Manual of Technical Guidelines against Establishing Sabo Master Plan for Debris Flows and Driftwood* , Part 2.6.5.

(3) Slope protection and scouring measures of slope toe of the embankment of a debris flow direction-controlling work

Embankment slope of the debris flow direction-controlling work is protected from debris flow erosion by revetment with concrete, stone masonry, concrete block, steel sheet pile, etc. The toe of the structure is protected against scouring by embedding revetment work, concrete block foot protection work, and foot protection groin work.

(4) Sediment and driftwood removal

Sediment and driftwood removal at a debris flow direction-controlling work is based on the Guideline, Section 3.

Section 3. Sediment and driftwood removal

> For the debris flow and driftwood countermeasure structure to fully function, sediment deposition shall be inspected periodically after debris flow occurrence. Then, sediment and driftwood removal shall be conducted as necessary.
>
> Furthermore, if sediment and driftwood removal is required in the debris flow and driftwood treatment plan, transportation method including route for carrying out shall be studied in advance.

Explanation

If sediment and driftwood removal is required in the debris flow and driftwood treatment plan, the transportation method, construction of transportation roads, receiver of transported debris, and frequency of sediment and driftwood removal shall be studied. Further, in principle, sediment and driftwood removal is not carried out for riverbed sediment stabilization work.

In addition, sediment and driftwood removal consists of "emergency sediment and driftwood removal" which is conducted urgently after debris flow occurs, and "periodic sediment and driftwood removal" where deposited sediment and driftwood is removed based on periodic inspection. The basic concept of sediment and driftwood removal is explained in Chapter 5 of the *Manual of Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* .

Section 4. Setting design external forces during a debris flow

4.1 Calculation of design external forces during a debris flow (the impact load is excluded)

The debris flow peak discharge, debris flow velocity and depth, unit weight of debris flow, and the drag force of debris flow are necessary to set the design external forces during a debris flow. They shall be calculated by assuming that no debris flow and driftwood facilities exists.

Explanation

The debris flow peak discharge is calculated based on Section 2.6.3, while the debris flow velocity and depth calculation is based on Section 2.6.5, the unit weight is based on Section 2.6.6, and the drag force of debris flow is based on Section 2.6.7 of the *Manual of Technical Guideline for Establishing Sabo Master Plan against Debris Flows and Driftwood* .

4.2 Impact load of boulders

The impact load received by the structure body due to the collision with a boulder varies according to the type and characteristics of structure body material. The impact load due to the collision of a boulder as a design external force is set in consideration of the type of material and characteristics of structure body.

Explanation

For mass concrete, the force (*P*) can be estimated by equation (11) [10), 11)].

$$P = \beta \cdot n\alpha^{3/2}, \qquad n = \sqrt{\frac{16R}{9\pi^2(K_1 + K_2)^2}}$$

$$K_1 = \frac{1 - {v_1}^2}{\pi E_1}, \qquad K_2 = \frac{1 - {v_2}^2}{\pi E_2}$$

$$\alpha = \left(\frac{5U^2}{4n_1 n}\right)^{2/5}, \qquad n_1 = \frac{1}{m_2}$$

$$\beta = (E + 1)^{-0.8}, \qquad E = \frac{m_2}{m_1} U_b^2 \quad \text{..(11)}$$

Where,

E_1, E_2 : elastic modulus of concrete and boulder (N/m^2), respectively,

v_1, v_2 : Poisson's ratio of concrete and boulder, respectively,

m_2 : weight of boulder (kg),

R : radius of boulder (m),

π : Pi (= 3.14),

U_b : velocity of boulder (m/s),

α : dent depth (m),

K_1, K_2 : constants,

β : experimental constant

m_1 : weight of concrete block (kg).

The velocity of boulder is considered to be equal to the debris flow velocity, and the boulder size is the maximum boulder diameter (see the Guideline, Section 2.1.3.1 (4)).

〜〜〜

(Appendix C) Example of physical constants of gravel and concrete [10)]

Elastic modulus of gravel: $E_2 = 5.0 \times 10^9 \times 9.81$ N/m^2, Poisson's ratio: $v_2 = 0.23$

Ultimate strength of static elastic modulus※ of concrete:

$E_1 = 0.1 \times 2.6 \times 10^9 \times 9.81$ N/m^2, Poisson's ratio of concrete: $v_1 = 0.194$

* Indentations are generated on the concrete surface due to the collision of boulder, hence the average deformation coefficient (ultimate strength of deformation modulus) until failure of concrete is used. This coefficient is approximately 1/10 of the concrete's elastic modulus.

〜〜〜

4.3 Impact load of driftwood

> The impact load received by the structure body due to the collision with driftwood varies according to the type and characteristics of structure body material. The impact load due to the collision of driftwood as a design external force is set in consideration of the type of material and characteristics of structure body.

Explanation

When a driftwood capturing work with a concrete structure such as wings is installed in the debris flow zone and when calculating the impact load received by the structure body due to the collision of driftwood for examining the stability of the structure and members such as wings, the formula for calculating the impact load received by the structure body due to the collision of a boulder shall be applied.

References

1) Ministry of Construction, River Bureau, Sediment Control Division, Sediment Control Department (1999): Debris Flow Risk River and Debris Flow Risk District Survey Regulations (Tentative), p. 17
2) K. Kawabe, S. Sakamoto, T. Uchida, and R. Itou(2014): Design of countermeasures for small rivers in Ohmachi, western Hiroshima Mountains, Journal of the Japan Society of Erosion Control Engineering, Vol. 67, No. 2, p. 42-46
3) Japan Society of Erosion Control Engineering (1996): Report by the Sabo Structure Seismic Design Study Committee, Journal of the Japan Society of Erosion Control Engineering, Vol. 48, No. 6 (203), p. 37
4) Y. Shimoda, T. Mizuyama, N. Ishikawa, and K. Furukawa (1992): Impact model tests and simulation analysis ofconcrete sabo dam sleeve under huge stone, Journal of the Japan Society of Civil Engineering, No. 450, p. 131-140
5) Y. Shimoda, S. Suzuki, N. Ishikawa, and K. Furukawa (1993): Impact failure analysis of concrete check dam by distinct element method, Journal of the Japan Society of Civil Engineering, No. 480, p. 97 - 106
6) M. Watanabe, T. Mizuyama, and S. Uehara (1980): Study of debris flow countermeasure Sabo structures, Journal of the Japan Society of Erosion Control Engineering, No. 115, p. 40
7) T. Mizuyama, S. Kobashi, and H. Mizuno (1995): Control of passing sediment with grid-type dams, Journal of the Japan Society of Erosion Control Engineering, Vol. 47, No. 5, p. 8 – 13
8) T. Mizuyama and S. Uehara (1981): Behavior of debris flows in bent channels, PWRI Technical Document, 23-5, p. 243
9) Ministry of Construction, River Bureau, Sediment Control Division, Sediment Control Department (1998): Guideline to preparing green Sabo zone plans (tentative), p. 5
10) K. Senoo, T. Mizuyama, and H. Shimohigashi (1985): Report on testing and analysis of buffer materials under shock of debris flow impact, Technical Memorandum of PWRI, No. 2169
11) T. Mizuyama and M. Imaki (1980): Experiments on the impact force of debris-flow against sabo dams, Civil Engineering Journal, Vol. 22, No. 11, p. 27 - 32

Appendix D Design of driftwood facilities in the bedload transport zone

Appendix 1.1 Scale of flood and sediment

When a driftwood countermeasure structure is constructed inside or near the river channel of a bedload transport zone, the structure shall be designed considering the scale of flood and immature debris flow so that those can be discharged safely.

Explanation

As a general rule, the scales of flood (peak discharge, flow velocity, depth and sediment content) produced by heavy rainfall are decided based on the *Technical Criteria for River Works (Tentative)* ; Practical Guide for Planning, the *Technical Criteria for River Works (Tentative)* ; Surveying, Chapter 3, and the *Technical Criteria for River Works (Tentative)* ; Design, Chapter 3.

The flow velocity and depth of a flood or of an immature debris flow are calculated based on Manning's equation using the discharge that includes sediment. Meanwhile, driftwood is not considered in the velocity and depth calculation. Further, the flow velocity of driftwood is assumed to be almost equal to the velocity of flow surface of a flood flow or of an immature debris flow, and it is calculated as about 1.2 times the average flow velocity.

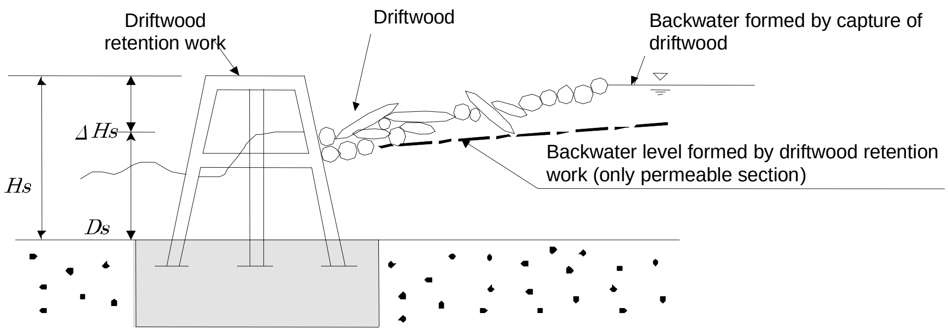
Appendix 1.2 Design of driftwood retention work

Appendix 1.2.1 Height of permeable section

The height of permeable section of a driftwood retention work shall be equal to or higher than the total of the water level after the occurrence of backwater effect formed by the driftwood retention work plus the additional height necessary to capture driftwood.

Explanation

The permeable section is designed so that it is not blocked by boulders, and the height of permeable section of a driftwood retention work shall be equal to or higher than the total of the water level after the occurrence of backwater effect formed by the driftwood retention work plus the additional height necessary to capture driftwood (see Appendix Figure 1 for the outline). The symbols in the figure are D_s: water level considering the backwater caused by the driftwood retention work (m), ΔH_s: height necessary to retain driftwood (m), and H_s: height of the driftwood retention work (permeable section). The procedure to determine the height of the permeable section and others is explained as follows.



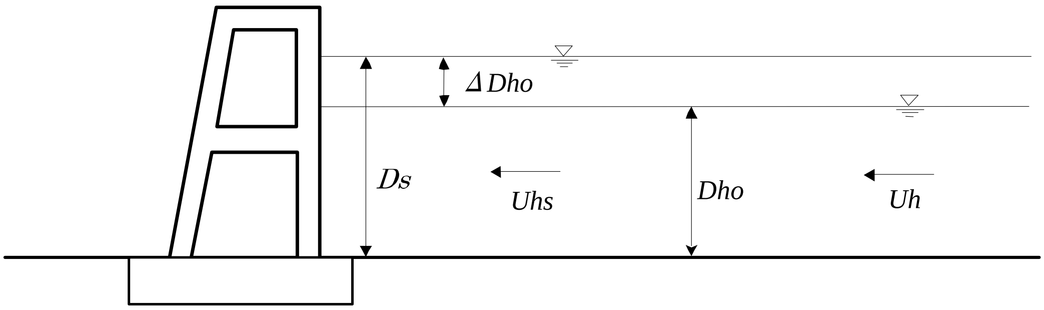
Appendix Figure 1 Schematic diagram of the height (H_s) of the permeable section of driftwood retention work installed in bedload zone

(1) Calculating the water level after the occurrence of backwater effect (D_s)

1) Depth before backwater D_{h0} and average flow velocity U_h are

At the channel section with weir : obtained by weir formula of weir using discharge of water containing fine sediments.

Otherwise: obtained by Manning's equation using discharge of water containing fine sediments.



Appendix Figure 2 Water level after the occurrence of backwater effect caused by a driftwood retention work

2) Water level rise due to backwater caused by a driftwood retention work

When driftwood retention work is installed in a fluvial descending zone, most driftwood will flow on the surface of flood and immature debris flow. Therefore, the height of the driftwood retention work must be higher than the flood and immature debris flow by considering the backwater caused by members consisting of the permeable section of the driftwood retention work.

The water level after the occurrence of backwater effect caused only by vertical members is calculated by the following equation [App. 1].

$$\Delta D_{h0} = k_m \cdot \sin\theta_m \cdot \left(\frac{R_m}{B_p}\right)^{4/3} \cdot \frac{U_h^{\,2}}{2g} \quad \text{(App. l)}$$

Where,

ΔD_{h0} : backwater height by the vertical members of the driftwood retention work (m),

k_m : coefficient depending on the shape of the cross-section of the vertical members (steel pipe $k_m \approx 2.0$, hollow square tube $k_m \approx 2.5$, and H-shaped beam $k_m \approx 3.0$),

θ_m : angle of the vertical members to the downstream river bed surface (degrees),

R_m : Projection width of vertical member (m),

B_p : spacing of the vertical members (m), and

U_h : flow velocity on the upstream side (m/s).

3) Water depth, D_s and average flow velocity U_{hs} after the occurrence of the backwater are shown below.

$$D_s = D_{h0} + \Delta D_{h0} \quad \text{(App. 2)}$$

$$U_{hs} = \frac{Q}{D_s \cdot B_s} \quad \text{(App. 3)}$$

Where,

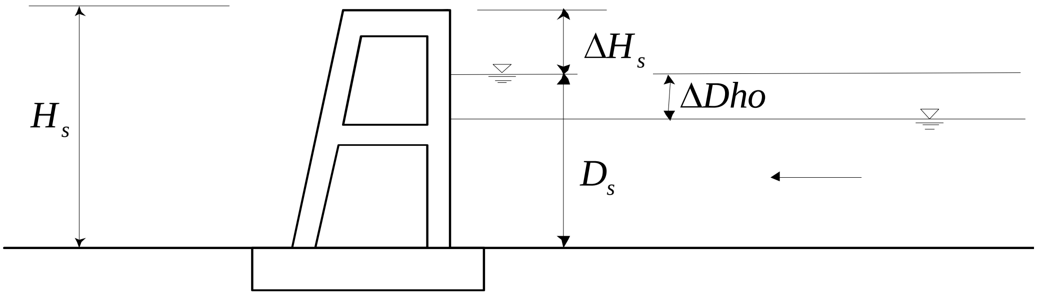
Q : design discharge (m^3/s),

U_{hs} : average flow velocity of backwater (m/s), and

B_s : flow width (m).

(2) Height of driftwood retention work (H_s)

The height of driftwood retention work is the total of the depth including the water depth after the occurrence of the backwater D_s plus the additional height necessary to capture driftwood ΔH_s, assuming that the driftwood retention work is not blocked by sediment and gravel. ΔH_s shall be set at least twice the maximum driftwood diameter considering the situation in which driftwood rides over each other when it is captured.



Appendix Figure 3 Height of the permeable section

Appendix 1.2.2 Spacing of members in the permeable section

The spacing of members on the permeable section of driftwood capturing works shall satisfy two conditions, i.e. the permeable section is not blocked by boulders and it captures driftwood.

Explanation

(1) Maximum boulder size transported by bed load

The maximum boulder size that flows in bedload zone is calculated by the following manner referring to the maximum movable boulder size by critical tractive force.

1) Square of the critical friction velocity to the average grain size: U_{*cm}^2

It is obtained by the following equation [App. 4].

$$U_{*cm}^2 = 0.05 \cdot \left(\frac{\sigma}{\rho} - 1\right) \cdot g \cdot d_m \quad \text{(App. 4)}$$

Where,

d_m : average diameter of riverbed material (m),

σ : density of boulder, generally from 2,600 to 2,650 kg/m^3,

ρ : density of muddy water, generally 1,000 to 1,200 kg/m^3, and

g : gravitational acceleration (m/s^2).

2) Square of the friction velocity: U_*^2

It is obtained by the following equation.

$$U_*^2 = g \cdot D_{h0} \cdot I \quad \text{(App. 5)}$$

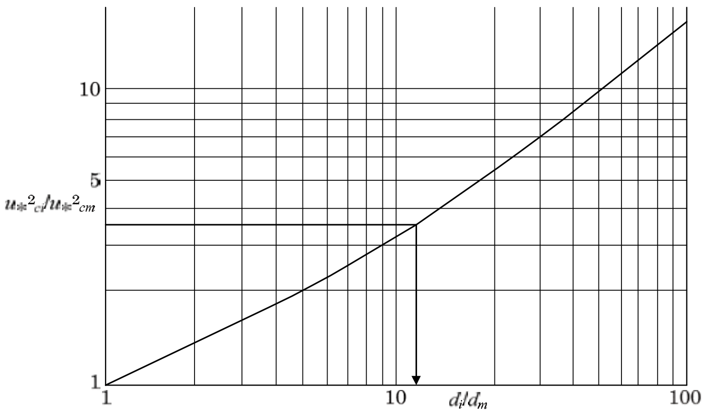
Where, D_{h0} : depth (m), I : riverbed gradient

3) Square of the shear velocity ratio: U_*^2/U_{*cm}^2

It is obtained using the values in 1) and 2).

4) Find the d_i/d_m for the point where the vertical axis U_*^2/U_{*cm}^2 in the figure is equal to U_*^2/U_{*cm}^2 in 3)

$$\frac{d_i}{d_m} > 0.4: \frac{U_{*ci}^2}{U_{*cm}^2} = \left(\frac{log_{10} 19}{log_{10}\left(19\frac{d_i}{d_m}\right)}\right)^2 \left(\frac{d_i}{d_m}\right) \quad \text{(App. 6)}$$



Appendix Figure 4 Critical tractive force by diameter

5) The maximum size is the smaller value between the calculated value above and the maximum size on site.

(2) Spacing of members on the permeable section

To avoid blockage of the permeable section by the boulders, the spacing of members is determined so that the maximum boulder size obtained above satisfies the following condition.

$$B_{wp} \geq 2d_i \quad \text{(App. 7)}$$

Where, B_{wp}: spacing of the permeable section (m), d_i: diameter of class *i* particles (m).

The spacing of the permeable section is also necessary to satisfy the following condition.

$$\frac{1}{2}L_{wm} \geq B_{wp} \quad \text{(App. 8)}$$

Where, L_{wm}: maximum driftwood length (m)

The spacing of members for capturing driftwood shall be the value that satisfies both of the conditions shown above.

Attention should be paid not to let the broken driftwood pass through the driftwood retention work.

Appendix 1.2.3 Examination of overall safety

> The stability of driftwood retention work shall be examined so that the designed structure is stable, even when it is fully blocked by driftwood.

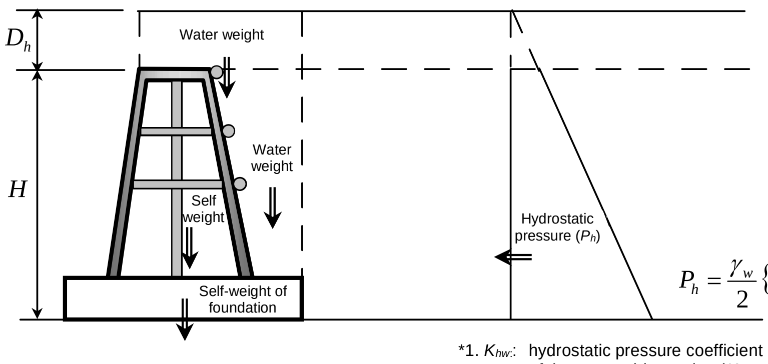
Explanation

The stability of a driftwood retention work in a bed load zone is examined based on the *Technical Criteria for River Works*: Practical Guide for Planning, the *Technical Criteria for River Works (Tentative)* , Design, Chapter 3.

The height of the driftwood retention work including the bottom concrete slab which is installed independently shall be 5 m or less (same as the height of a groundsill). However, if the height exceeds 5 m, stability of the structure shall be examined by considering the following points:

- Widen the spillway and reduce the water depth to minimize the height of the permeable section of driftwood retention work.
- If the bottom concrete slab is thick and a large gap is formed between top of the bottom concrete slab and the downstream riverbed surface, or if the driftwood retention work is high and a large gap is formed at the overflow section, downstream riverbed protection work shall be considered to ensure stability.

In bedload zone, if the driftwood retention work is blocked by driftwood, hydrostatic pressure acts as shown in Appendix Figure 5. The magnitude of hydrostatic pressure is affected by the blockage ratio (K_{hw}) of the permeable section. Further, if the permeable section is completely blocked, the hydrostatic pressure coefficient is assumed to be K_{hw} = 1.0 (unit weight of water γ_w = 11.77 kN/m^3). For a permeable type driftwood retention work in the bedload zone, sediment pressure is not considered because boulders are not designed to be captured.



$$P_h = \frac{\gamma_w}{2}\{(H + D_h)\cdot K_{hw}\}^2$$

*1. K_{hw}: hydrostatic pressure coefficient according to blockage ratio of the permeable section (K_{hw} =1.0)

Appendix Figure 5 Blockage of driftwood retention work in a bedload zone

Reference Table 1 Design external forces (excluding self-weight) of a driftwood countermeasure structure (Bedload zone)

	Normal time	During debris flow	During flood
Dam height of 5 m or less (including bottom concrete slab)			Hydrostatic pressure

Appendix 1.2.4 Examination of stability of members of permeable section

Members of the permeable section of driftwood retention work in bedload zone shall be examined to verify their stability against water pressure and against the impact load of driftwood and boulder.

Explanation

Similar to driftwood retention work in a debris flow zone, the cross-section composing the permeable section is small and not a gravity type structure. Thus, structural calculations of the members are conducted to verify their safety.

The impact load due to collision of driftwood and boulder shall comply with the *Manual of Technical Guideline for designing Sabo Facilities against Debris flow and Driftwood* , Section 4.2 and 4.3.

For calculating the impact load of driftwood as design external force which used for structural calculations of permeable members in a bedload zone, the surface flow velocity is applied as shown in the equation below while the average flow velocity is applied for calculating the impact load of boulders. The impact load is calculated assuming that the long axis of the driftwood flows down parallel to the water flow direction and collides with the driftwood retention work.

$$U_{ss} = 1.2U_s \qquad \text{(App. 9)}$$

Where, U_{ss}: surface flow velocity (m/s) and U_s: average flow velocity (m/s).

Appendix 1.2.5 Design of parts other than the permeable section

The structure of all members of a driftwood retention work shall be designed so that the driftwood retention work is stable even when it is blocked with driftwood. Its stability against the impact load due to collision of driftwood also need to be examined.

Explanation

The structures of members (spillway section, crest width, downstream slope, ground, structure of wings, downstream riverbed protection work (apron)) of driftwood retention work are examined based on the *Technical Criteria for River Works*: Practical Guide for Planning, the *Technical Criteria for River Works (Tentative)*, Design, Chapter 3. This implies that examination of the structures of driftwood retention work is conducted by assuming that the upstream side of the driftwood retention work (permeable section) is completely blocked by driftwood and water cannot pass through. Thus, the driftwood retention work is considered as an impermeable type Sabo dam when designing the spillway section, crest width, downstream slope, ground, and downstream riverbed protection work. In addition, constructing the driftwood retention work as a sub-dam to the Sabo dam is also possible.

The spillway section of a driftwood retention work is constructed above the permeable section to prepare for any overflow of sediment flow and flood due to blockage of the permeable sections by driftwood. Freeboard is not required, since water can penetrate from the crest of the permeable structure.

Appendix 1.3 Design of driftwood restraint work

Driftwood restraint work in bedload zone shall be designed so that it efficiently prevents erosion of riverbanks and is safe against floods.

Explanation

Driftwood restraint work in bedload zone is constructed to functions similarly and at the same locations as revetment works for embankment and channel works. The design is based on the *Technical Criteria for River Works (Tentative)*, Design, Chapter 3.

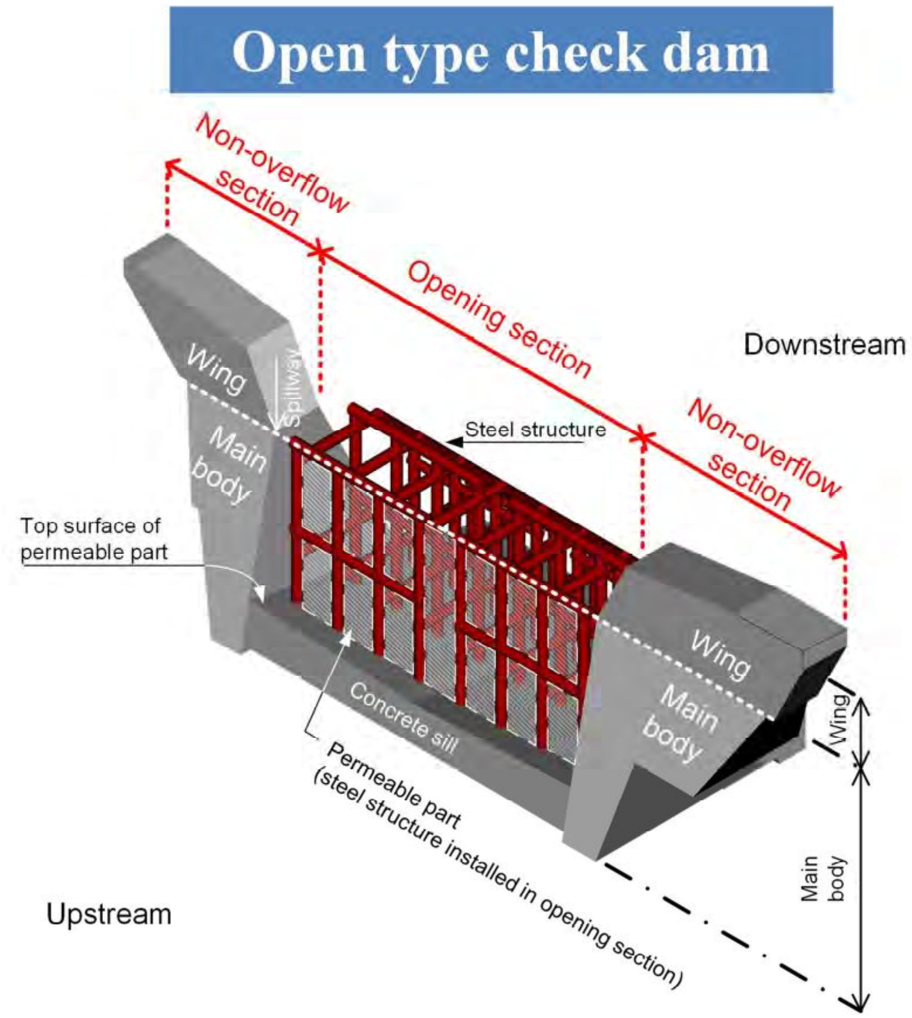
References of Appendix D

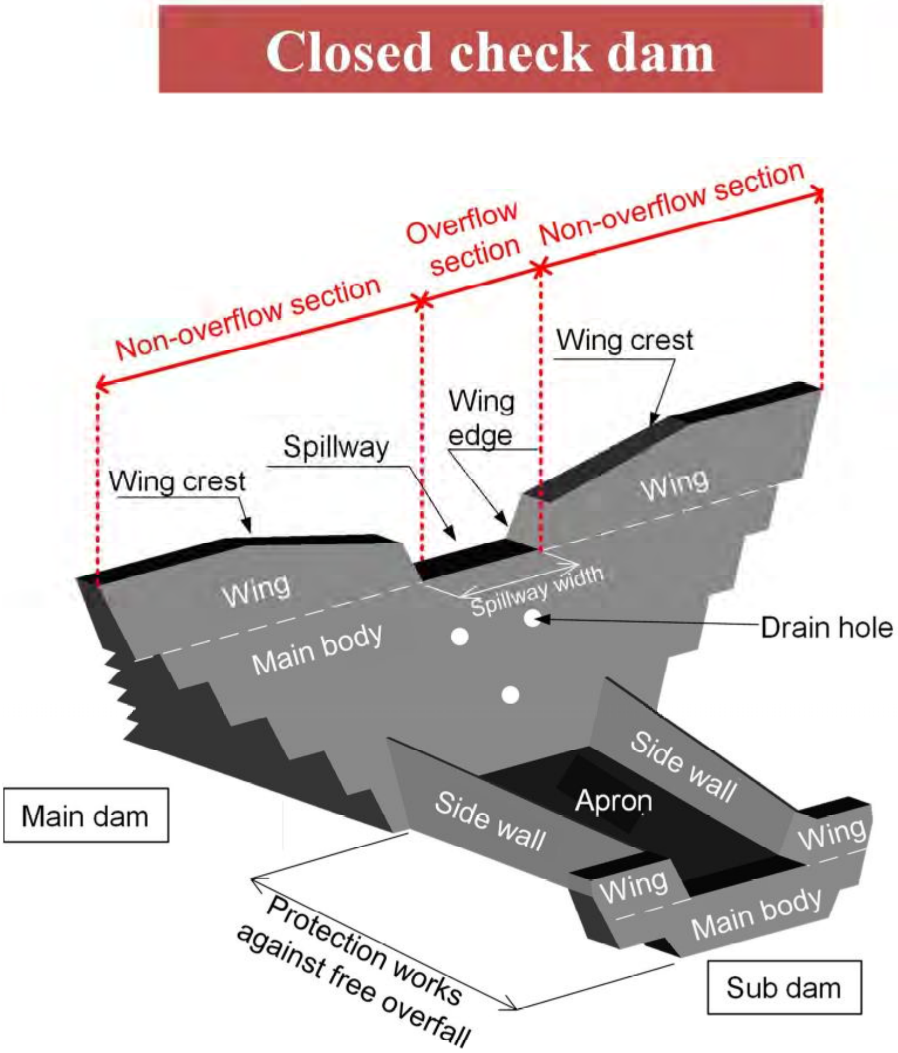
Reference 1) Japan Society of Civil Engineering (1971): Hydraulic Equations, 1971 Revised Edition, Japan Society of Civil Engineering, p. 252

Reference 2) Japan Society of Civil Engineering (1999): Hydraulic Equations, 1999 Revised Edition, Japan Society of Civil Engineering, p. 158

Appendix E Parts names of check dams

Courtesy of Public Works Research Institute




Closed check dam
Non-overflow section
Overflow section
Non-overflow section
Wing crest
Wing edge
Spillway
Wing crest
Wing
Wing
Spillway width
Drain hole
Main body
Side wall
Apron
Side wall
Main dam
Wing
Wing
Main body
Protection works against free overfall
Sub dam

Appendix F List of symbols

B	channel width (m)
B_1	base width of the spillway (m)
B_2	width of the overflow (m)
B_{d1}	width of the upstream channel of debris flow depositing area (m)
B_{d2}	width of the depositing area of debris flow depositing area (m)
B_p	spacing of the permeable section of open type Sabo dam (m)
B_r	channel width (m)
B_s	flow width (m)
B_{wp}	spacing of the vertical members of driftwood retention work (m)
C	coefficient of discharge (0.6 to 0.66)
C_*	volumetric concentration of riverbed sediment
C_e	coefficient of earth pressure
D_d	depth of debris flow (m)
$\triangle D_d$	freeboard height (m)
D_{ds}	depth in the straight section (m)
D_h	overflow depth (m)
D_{h0}	water depth before backwater (m)
ΔD_{h0}	backwater height by the vertical members of the driftwood retention work (m)
d_i	diameter of class *i* particles (m)
d_m	average diameter of riverbed material (m)
D_s	water level considering the backwater caused by the driftwood retention work (m)
D_{95}	grain size for which 95% of the material weight is finer (treated as the maximum boulder size) (m)
E_1, E_2	elastic modulus of concrete and boulder (N/m^2)
F	drag force of debris flow
g	gravitational acceleration (9.81 m/s^2)
H	Sabo dam height (m)
H_c	height of impermeable part of semi-open type Sabo dam (m)
H_s	height of driftwood retention work (m)
$\triangle H_s$	height necessary to retain driftwood (m)
I	riverbed gradient
K_1, K_2	constants for P
K_{hw}	blockage ratio of the permeable section
k_m	coefficient depending on the shape of the cross-section of the vertical members
L	horizontal length of downstream slope of closed type Sabo dam

L_{min}	smaller one of horizontal span and vertical span in a grid type Sabo dam (m)
L_{wm}	the maximum driftwood length (m)
m_1	weight of concrete block (kg)
m_2	weight of boulder (kg)
P	Impact force of boulder or driftwood (kN)
P_{sed}	peak sediment load at the downstream end of a channel with a structure (m^3 / sec)
P_{sed0}	peak sediment load at the downstream end of a channel without a structure (m^3 / sec)
Q	discharge of water containing fine sediments (m^3/s)
Q	design discharge (m^3/s)
R	radius of boulder (m)
R	radius of curvature at the center of the channel (m)
$R_{(IN)}$	radius of curvature of the bend (m)
R_m	Projection width of vertical member (m)
U	average flow velocity (m/s)
U_*	shear velocity (m/s)
U_b	velocity of boulder (m/s)
U_{*cm}	velocity corresponding to the beginning of motion of average diameter (m/s)
U_h	flow velocity on the upstream side (m/s)
U_{hs}	average flow velocity of backwater (m/s)
U_s	average flow velocity (m/s)
U_{ss}	surface flow velocity (m/s)
v_1, v_2	Poisson's ratio of concrete and boulder
V_c	dam body block volume (m^3) calculated by assuming the overflow section as a closed structure
W	Self-weight of the wing (kg)
W_{rc}	dam body block weight (kN) calculated by assuming the overflow section as an open structure with concrete
α	dent depth (m)
β	experimental constant for P
γ_d	unit weight of the debris flow (kN/m^3)
γ_{rc}	apparent unit weight of concrete (kN/m^3)
γ_s	unit weight of sediment in water (kN/m^3)
γ_W	unit weight of water
π	pi (=3.14)
θ_c	slope of the debris flow direction-controlling work to flow direction (degrees)
θ_m	angle of the vertical members to the downstream riverbed surface (degrees)
θ_{f1}	eccentric angle to the sabo dam (degrees)
θ_{f2}	angle of the hypothetical stream axis of the debris flow and the axis perpendicular to the dam axis (degrees)

θ_{f3}	allowable angle (degrees)
ρ	allowable angle (degrees)

TECHNICAL NOTE of NILIM

NO.905 May 2022

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Ministry of Land, Infrastructure, Transport and Tourism

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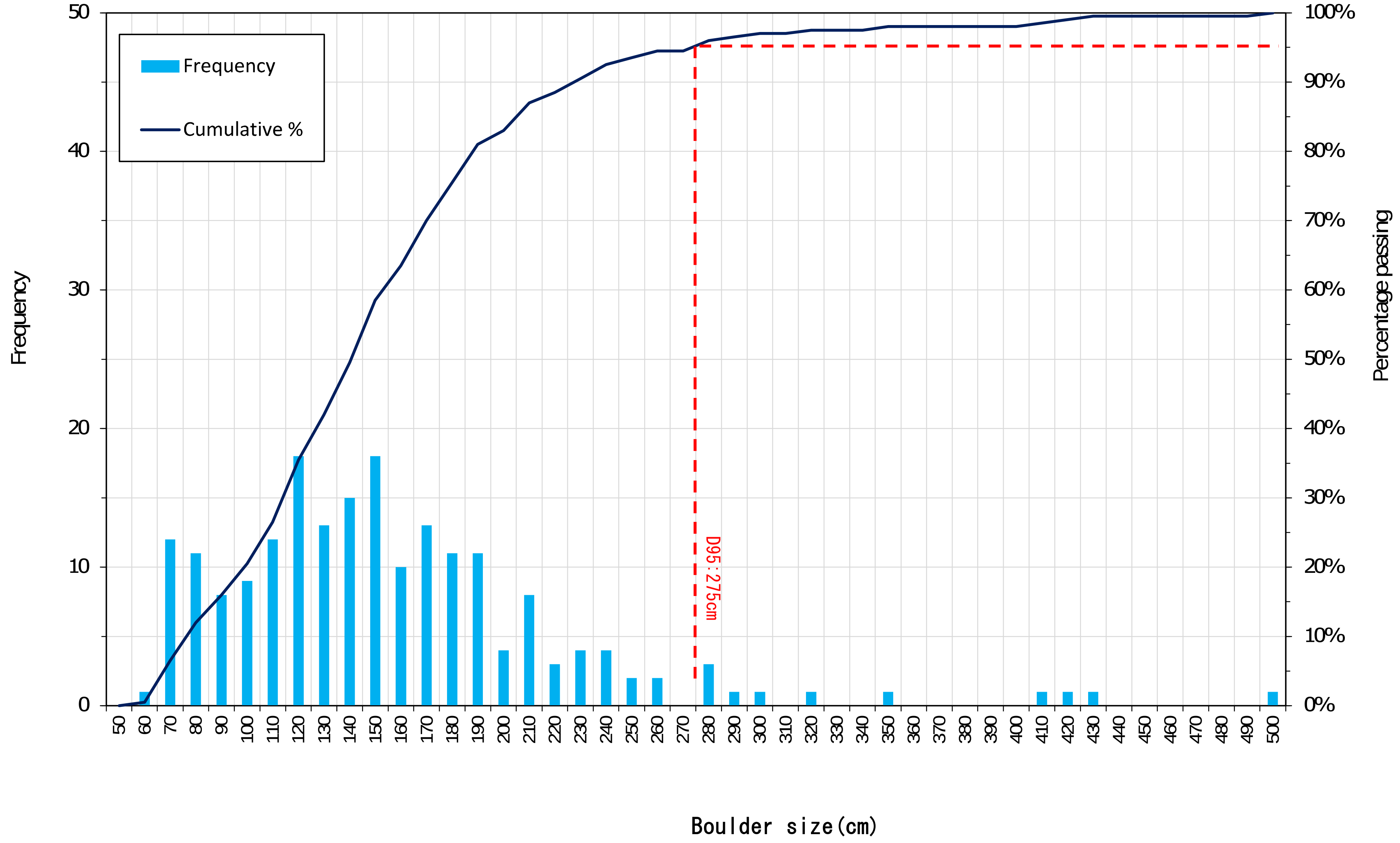
Appendix 4-2　Boulder Size Survey

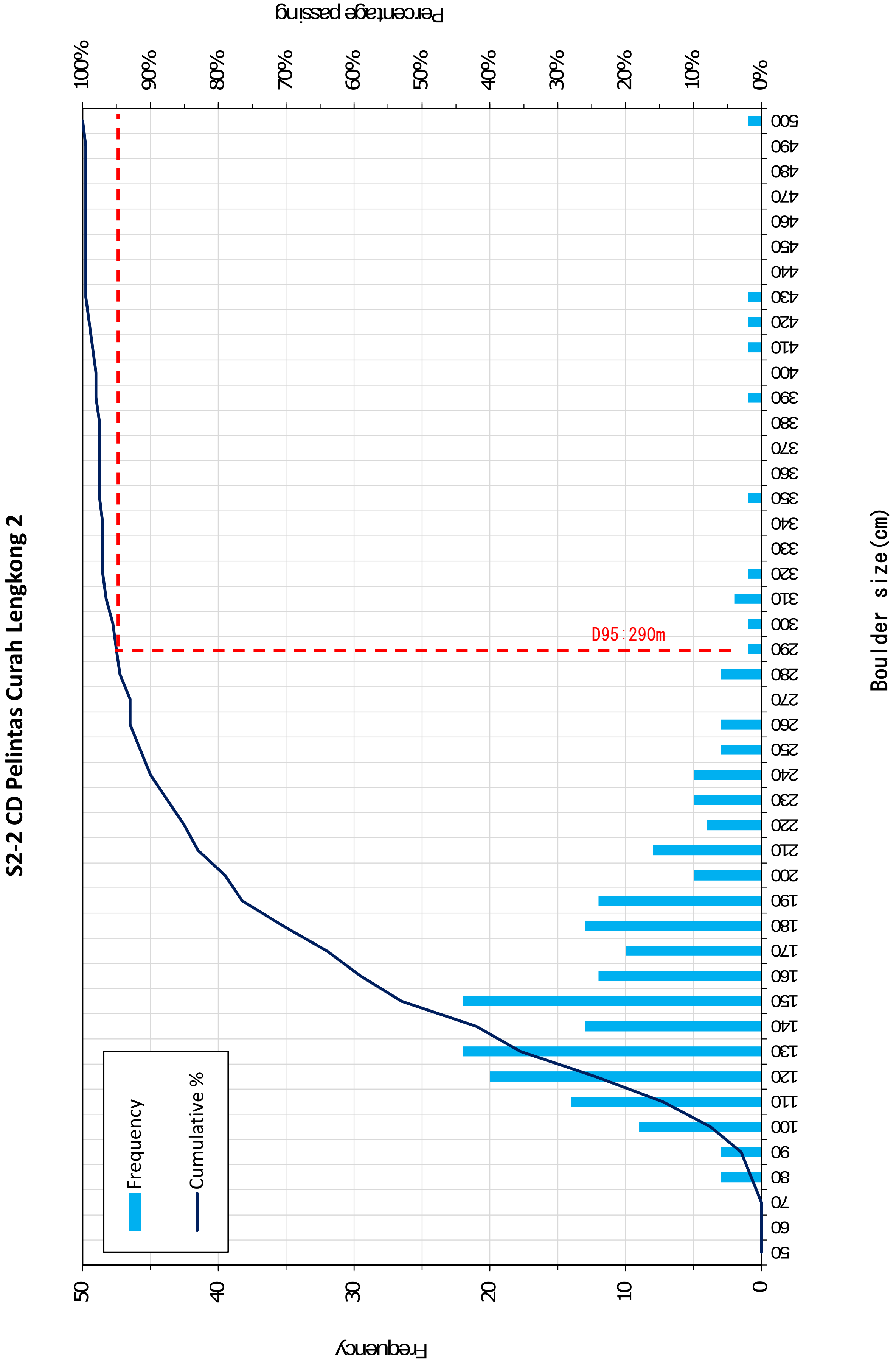
Table Result of Boulder Size（D_{95}）

Target Area	Boulder Size (D_{95})	Average Size
CD Curah Kobokan5	2.75m	2.5m
CD Pelintas Curah Lengkong 2	2.90m	
DD Leprak 2	2.45m	
DD Leprak 3	2.40m	
KD Leprak 3	2.30m	

The boulder size for this project will be 2.5 meters.

S2-1 CD Curah Kobokan 5




S2-2 CD Pelintas Curah Lengkong 2
Percentage passing
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Frequency
50
40
30
20
10
0
Frequency
Cumulative %
D95:290m
Boulder size(cm)
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290
300
310
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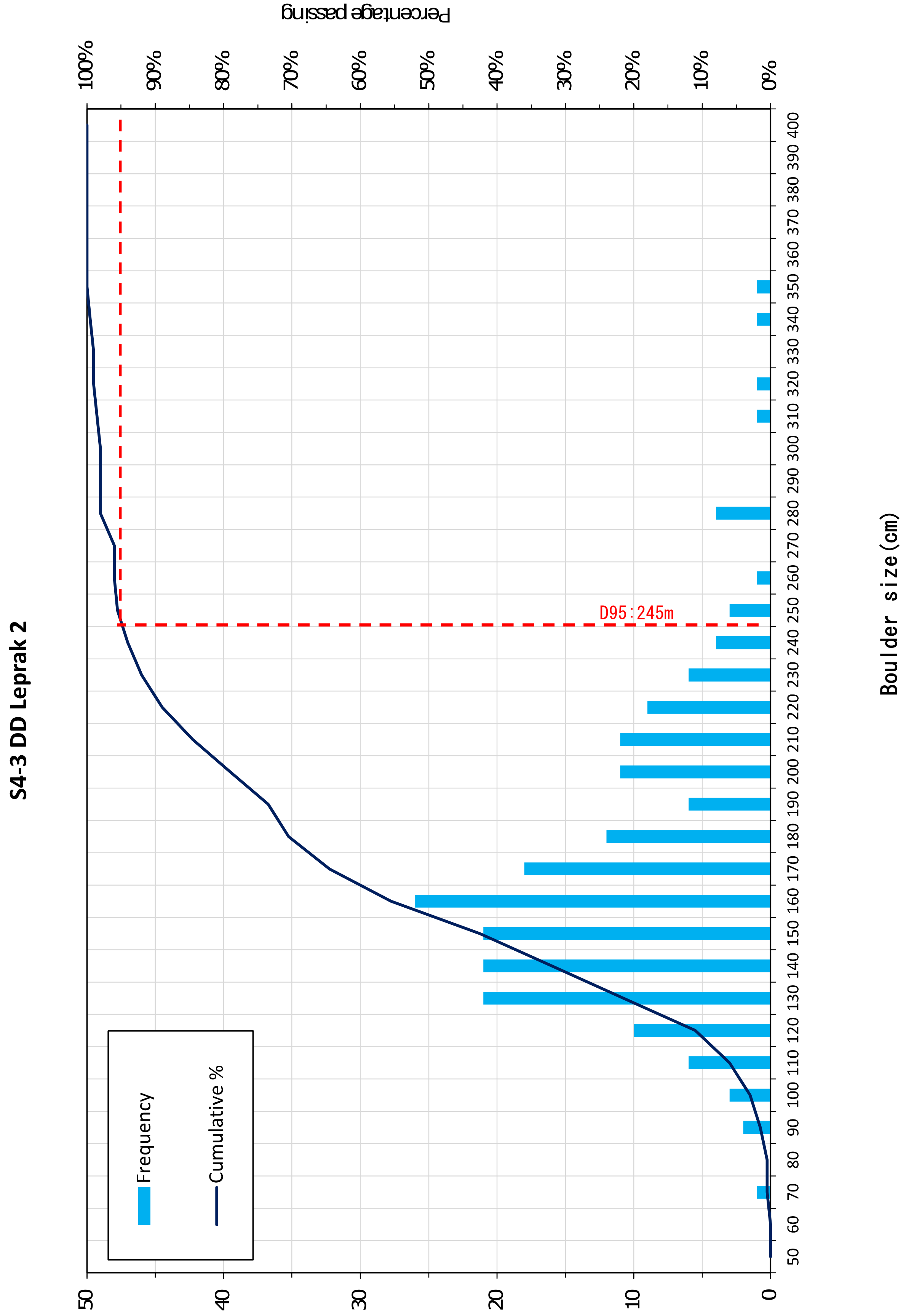
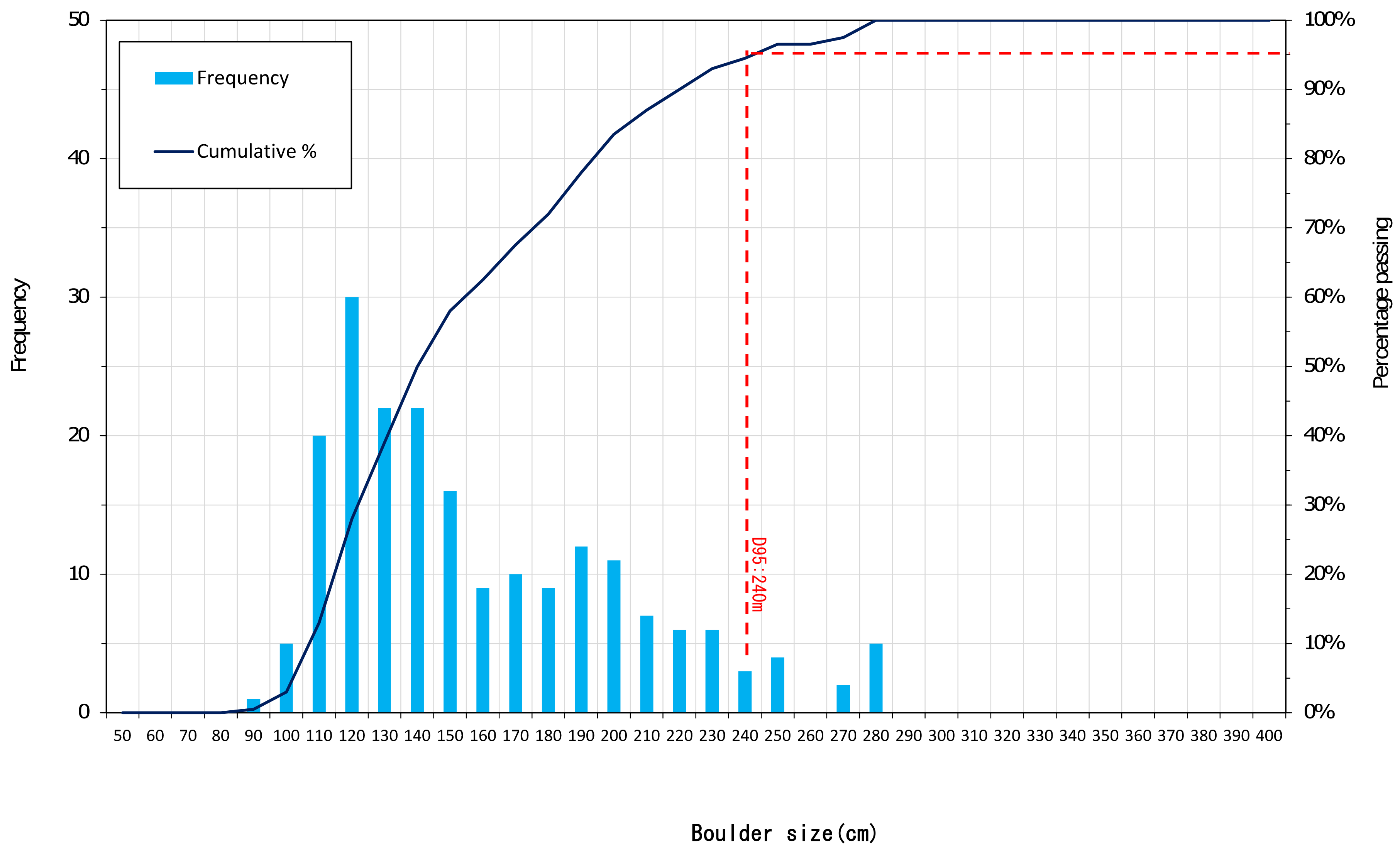
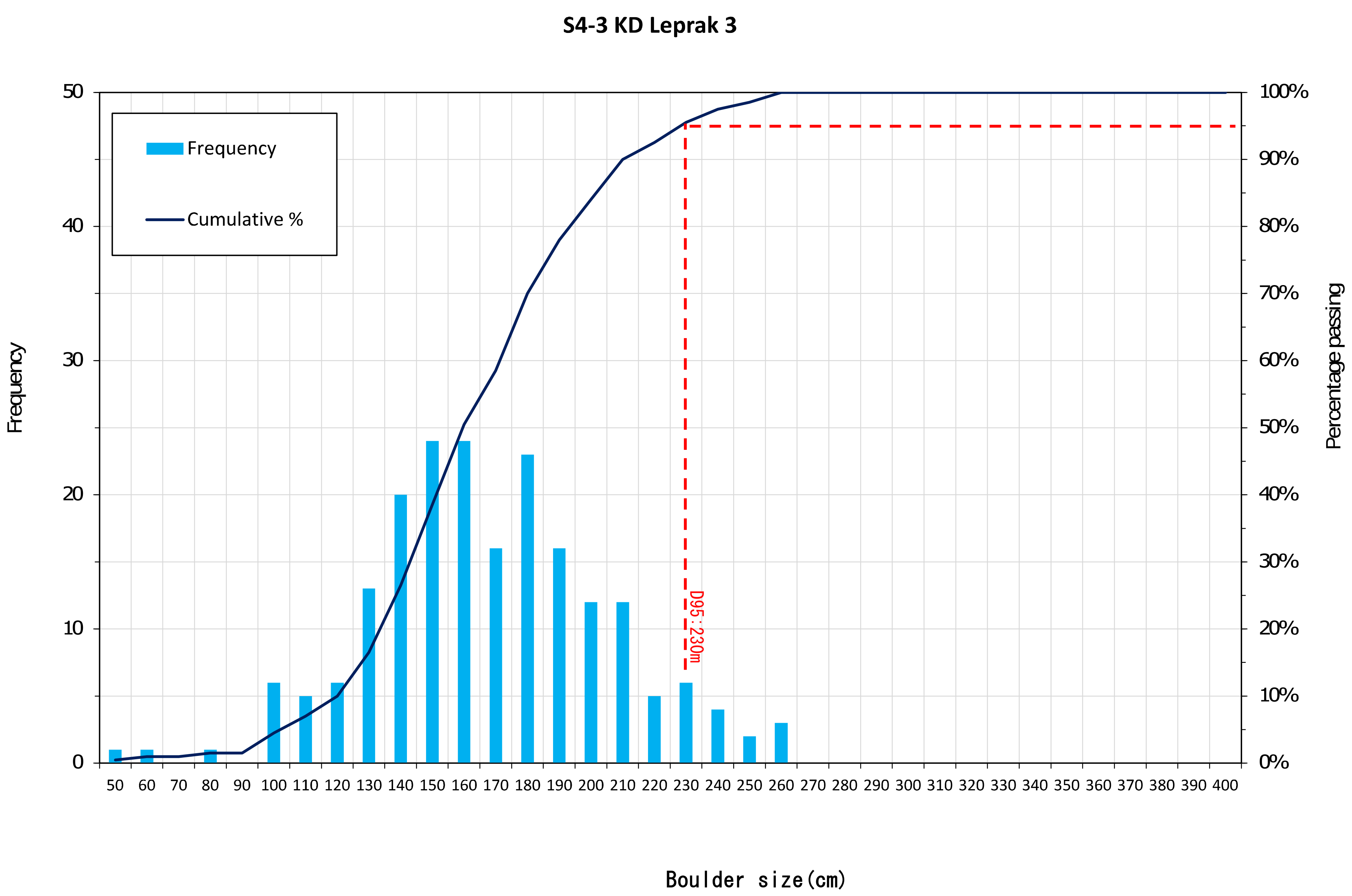
460
470
480
490
500


S4-3 DD Leprak 2
Frequency
Cumulative %
D95:245m
Frequency
Percentage passing
Boulder size(cm)
50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250 260 270 280 290 300 310 320 330 340 350 360 370 380 390 400
0 10 20 30 40 50
0% 10% 20% 30% 40% 50% 60% 70% 80% 90% 100%

S4-3 DD Leprak 3




S4-3 KD Leprak 3
Frequency
Cumulative %
D95:230m
Frequency
Percentage passing
Boulder size(cm)
0
10
20
30
40
50
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%
50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250 260 270 280 290 300 310 320 330 340 350 360 370 380 390 400

Boulder Size Survey

Target Area D01 (upstream of CD Kobokan 5)

Date 2024/1/25

Person in charge Indra Karya

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	80	60	120	83
2	80	50	70	65
3	150	95	55	92
4	60	45	100	65
5	95	45	100	75
6	100	90	110	100
7	70	65	95	76
8	140	110	120	123
9	80	70	60	70
10	80	60	95	77
11	140	160	110	135
12	105	90	70	87
13	85	100	100	95
14	45	80	85	67
15	70	55	65	63
16	115	120	105	113
17	105	65	60	74
18	185	90	160	139
19	130	100	85	103
20	110	40	90	73
21	95	70	105	89
22	85	60	70	71
23	80	55	85	72
24	210	160	110	155
25	180	120	200	163
26	120	50	85	80
27	75	65	75	72
28	115	85	125	107
29	165	100	195	148
30	105	70	125	97
31	140	100	130	122
32	175	165	110	147
33	95	70	100	87
34	140	110	135	128
35	190	145	170	167
36	210	140	150	164
37	75	65	55	64
38	120	75	125	104
39	210	160	150	171
40	150	85	130	118
41	180	155	160	165
42	145	110	135	129
43	175	110	135	137
44	135	85	155	121
45	175	170	190	178
46	120	150	130	133
47	140	110	165	136
48	250	210	190	215
49	85	70	140	94
50	90	45	100	74

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	170	110	145	139
52	235	160	170	186
53	120	110	115	115
54	135	130	155	140
55	50	75	85	68
56	90	75	120	93
57	125	80	65	87
58	170	165	130	154
59	80	50	85	70
60	185	165	150	166
61	100	75	105	92
62	80	60	55	64
63	140	60	155	109
64	170	115	125	135
65	145	60	155	110
66	180	110	60	106
67	180	130	125	143
68	60	80	55	64
69	230	150	175	182
70	120	140	110	123
71	125	110	125	120
72	75	55	85	71
73	169	150	155	158
74	110	195	115	135
75	175	120	170	153
76	95	80	95	90
77	185	100	90	119
78	70	65	70	68
79	150	80	105	108
80	120	110	105	111
81	50	55	70	58
82	190	155	145	162
83	120	90	115	107
84	280	135	140	174
85	120	110	95	108
86	115	270	160	171
87	75	60	70	68
88	160	110	140	135
89	150	120	110	126
90	120	100	95	104
91	220	190	290	230
92	120	110	85	104
93	310	220	180	231
94	130	85	75	94
95	90	90	65	81
96	135	130	80	112
97	140	120	80	110
98	220	220	180	206
99	110	95	95	100
100	105	85	70	85

Boulder Size Survey

Target Area	BD02 (downstream of CD Kobokan 5 and CD pelintas	Date	2024/1/25
		Person in charge	Indra Karya

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	195	130	120	145
2	180	120	110	133
3	184	165	105	147
4	235	155	165	182
5	125	95	115	111
6	115	105	100	106
7	220	175	165	185
8	210	170	155	177
9	280	255	185	236
10	135	100	145	125
11	150	120	105	124
12	135	110	115	120
13	165	105	95	118
14	215	125	105	141
15	190	125	215	172
16	155	85	120	116
17	270	190	220	224
18	130	105	110	115
19	260	145	175	188
20	195	130	150	156
21	175	135	120	142
22	255	180	110	172
23	170	125	110	133
24	185	100	105	125
25	195	140	120	149
26	175	135	105	135
27	180	150	140	156
28	185	110	155	147
29	140	105	100	114
30	180	125	135	145
31	245	155	160	182
32	200	135	125	150
33	215	160	135	167
34	345	295	220	282
35	285	200	155	207
36	185	115	100	129
37	210	120	170	162
38	210	170	185	188
39	220	120	225	181
40	260	155	135	176
41	195	185	160	179
42	680	360	300	419
43	340	230	195	248
44	360	340	250	313
45	370	310	230	298
46	275	235	170	222
47	350	265	160	246
48	255	215	220	229
49	320	240	205	251
50	270	210	180	217

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	410	490	395	430
52	190	145	110	145
53	300	260	265	274
54	320	250	270	278
55	350	270	210	271
56	450	410	370	409
57	300	250	210	251
58	330	190	210	236
59	470	430	600	495
60	390	350	300	345
61	210	150	110	151
62	260	220	145	202
63	310	240	170	233
64	180	150	120	148
65	260	240	170	220
66	240	150	145	173
67	135	110	100	114
68	210	170	170	182
69	190	130	180	164
70	240	230	140	198
71	140	90	180	131
72	130	110	120	120
73	130	110	100	113
74	175	115	140	141
75	165	120	105	128
76	185	130	110	138
77	140	85	110	109
78	200	170	130	164
79	280	230	140	208
80	180	140	120	145
81	150	110	190	146
82	150	130	110	129
83	210	130	150	160
84	170	170	150	163
85	200	165	235	198
86	220	200	150	188
87	180	185	110	154
88	265	165	190	203
89	225	105	130	145
90	210	210	185	201
91	165	130	150	148
92	185	100	250	167
93	190	125	180	162
94	230	200	165	197
95	175	220	200	197
96	275	200	165	209
97	220	180	145	179
98	160	225	185	188
99	165	170	130	154
100	240	200	175	203

Boulder Size Survey

Date 2024/1/25

Target Area BD03 (upstream of CD pelintas Curah Lengkong 2)

Person in charge Indra Karya

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	120	90	35	72
2	110	90	60	84
3	180	90	100	117
4	160	90	80	105
5	190	145	80	130
6	300	170	135	190
7	395	340	210	304
8	230	170	120	167
9	340	200	170	226
10	130	90	70	94
11	210	170	100	153
12	220	200	120	174
13	180	130	80	123
14	210	160	130	163
15	190	155	100	143
16	150	120	80	113
17	230	150	90	146
18	235	185	95	160
19	340	315	135	244
20	180	70	80	100
21	240	170	150	183
22	190	165	130	160
23	260	295	375	306
24	160	100	120	124
25	170	120	140	142
26	110	150	105	120
27	110	80	125	103
28	210	90	105	126
29	150	90	130	121
30	180	120	140	145
31	145	135	115	131
32	110	90	100	100
33	140	110	60	97
34	155	120	110	127
35	100	60	100	84
36	170	90	140	129
37	170	90	140	129
38	140	110	90	111
39	120	90	110	106
40	170	120	120	135
41	110	100	80	96
42	120	100	110	110
43	120	90	80	95
44	210	170	160	179
45	160	150	80	124
46	170	160	130	152
47	100	85	70	84
48	120	80	120	105
49	90	70	70	76
50	140	110	90	111

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	100	75	65	79
52	140	100	110	115
53	150	150	90	127
54	120	100	90	103
55	225	160	165	181
56	130	70	100	97
57	570	290	345	385
58	110	100	90	100
59	140	120	120	126
60	120	90	100	103
61	160	150	110	138
62	230	280	200	234
63	200	180	170	183
64	130	100	120	116
65	280	200	210	227
66	270	190	160	202
67	170	110	100	123
68	230	140	110	152
69	150	130	80	116
70	260	240	160	215
71	200	110	170	155
72	160	100	100	117
73	210	100	80	119
74	270	180	110	175
75	150	160	90	129
76	180	150	100	139
77	200	120	90	129
78	160	150	110	138
79	240	160	130	171
80	160	130	140	143
81	160	110	150	138
82	150	80	110	110
83	120	90	70	91
84	120	70	120	100
85	340	270	150	240
86	150	100	125	123
87	120	100	125	114
88	200	120	140	150
89	140	120	105	121
90	110	120	170	131
91	110	80	120	102
92	210	215	120	176
93	150	110	100	118
94	90	90	105	95
95	210	120	150	156
96	120	80	110	102
97	205	165	170	179
98	310	170	185	214
99	190	110	160	150
100	310	230	245	259

Boulder Size Survey

Target Area BD04

Date 2024/1/25

Person in charge Yokokura, Sultan, Leem, Irfan, Arif, Hefryan

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	360	360	300	339
2	220	180	110	163
3	280	160	100	165
4	200	150	90	139
5	190	170	180	180
6	190	160	180	176
7	200	130	110	142
8	220	190	190	200
9	300	320	210	272
10	210	160	130	163
11	200	200	170	189
12	190	140	140	155
13	180	150	150	159
14	300	180	190	217
15	290	130	130	170
16	310	210	150	214
17	310	220	210	243
18	180	160	120	151
19	230	150	150	173
20	230	170	170	188
21	230	210	180	206
22	300	170	170	205
23	360	280	280	304
24	260	280	170	231
25	240	180	180	198
26	20	180	150	81
27	200	170	120	160
28	180	180	120	157
29	360	240	230	271
30	360	310	280	315
31	200	200	100	159
32	180	160	160	166
33	380	310	180	277
34	320	220	220	249
35	240	180	140	182
36	220	140	120	155
37	120	120	140	126
38	120	120	140	126
39	120	140	140	133
40	180	160	120	151
41	200	120	160	157
42	180	100	100	122
43	200	120	160	157
44	160	100	120	124
45	150	120	120	129
46	180	100	140	136
47	180	100	120	129
48	200	140	140	158
49	180	140	140	152
50	140	140	80	116

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	120	120	120	120
52	100	140	120	119
53	160	110	100	121
54	150	100	110	118
55	140	120	120	126
56	200	120	120	142
57	300	140	200	203
58	140	120	100	119
59	160	120	140	139
60	150	120	100	122
61	120	100	100	106
62	340	200	240	254
63	190	140	160	162
64	180	160	140	159
65	220	160	140	170
66	200	150	150	165
67	150	100	70	102
68	120	100	80	99
69	150	100	80	106
70	180	120	70	115
71	170	160	120	148
72	200	150	70	128
73	160	110	70	107
74	280	200	150	203
75	310	190	150	207
76	130	100	60	92
77	190	130	90	131
78	180	150	120	148
79	220	150	80	138
80	370	190	160	224
81	460	300	290	342
82	220	160	80	141
83	290	240	120	203
84	210	130	80	130
85	210	140	80	133
86	250	140	100	152
87	270	180	160	198
88	110	80	60	81
89	140	130	70	108
90	310	190	100	181
91	230	180	130	175
92	260	200	170	207
93	360	230	170	241
94	270	180	150	194
95	190	160	90	140
96	190	120	70	117
97	200	140	130	154
98	120	90	70	91
99	220	210	100	167
100	280	210	130	197

Boulder Size Survey

Target Area	BD5
Date	2024/1/26
Person in charge	Leem, Irfan

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	220	180	200	199
2	170	170	150	163
3	240	200	220	219
4	200	220	190	203
5	260	190	160	199
6	260	200	220	225
7	240	180	200	205
8	210	190	190	196
9	380	160	220	237
10	240	160	240	210
11	120	160	160	145
12	200	140	100	141
13	120	120	130	123
14	160	100	180	142
15	120	140	180	145
16	180	140	120	145
17	130	110	160	132
18	140	120	180	145
19	180	180	160	173
20	160	140	160	153
21	160	120	140	139
22	140	160	130	143
23	120	110	160	128
24	130	140	150	140
25	200	100	160	147
26	180	140	140	152
27	170	150	140	153
28	180	110	100	126
29	140	120	130	130
30	200	120	140	150
31	140	160	120	139
32	130	150	140	140
33	120	100	100	106
34	130	110	110	116
35	180	140	180	166
36	140	140	160	146
37	200	160	160	172
38	180	180	200	186
39	160	140	100	131
40	150	150	120	139
41	140	100	140	125
42	130	90	140	118
43	140	100	100	112
44	140	130	120	130
45	120	120	140	126
46	220	160	220	198
47	200	160	180	179
48	220	160	120	162
49	230	120	180	171
50	180	140	160	159

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	300	170	200	217
52	200	160	120	157
53	160	140	150	150
54	260	140	220	200
55	150	160	140	150
56	120	120	140	126
57	130	150	160	146
58	200	160	160	172
59	260	160	240	215
60	220	200	140	183
61	300	180	180	213
62	240	120	140	159
63	180	120	100	129
64	160	110	130	132
65	200	90	140	136
66	180	100	120	129
67	170	12	120	63
68	180	180	140	166
69	170	150	160	160
70	170	110	120	131
71	200	140	140	158
72	240	200	240	226
73	360	160	180	218
74	170	150	180	166
75	260	200	260	238
76	200	140	160	165
77	220	100	140	145
78	200	120	140	150
79	200	140	150	161
80	220	120	160	162
81	210	150	140	164
82	180	120	180	157
83	220	220	250	230
84	240	140	160	175
85	150	160	160	157
86	140	140	120	133
87	300	200	200	229
88	220	210	160	195
89	380	180	200	239
90	200	180	200	193
91	180	140	180	166
92	160	140	120	139
93	160	120	140	139
94	240	120	180	173
95	200	120	140	150
96	300	180	180	213
97	180	120	180	157
98	250	180	240	221
99	360	200	300	278
100	200	220	240	219

Boulder Size Survey

Target Area BD06

Date 2024/1/26

Person in charge Mr. Yokokura and Sultan

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	175	85	140	128
2	180	170	140	162
3	115	170	140	140
4	180	340	240	245
5	220	140	175	175
6	340	160	285	249
7	120	175	125	138
8	100	200	160	147
9	100	160	120	124
10	100	115	120	111
11	200	170	90	145
12	110	110	100	107
13	200	130	130	150
14	190	220	110	166
15	130	160	150	146
16	160	160	120	145
17	110	160	140	135
18	200	170	90	145
19	355	240	150	234
20	240	200	150	193
21	210	130	110	144
22	210	200	160	189
23	150	200	250	196
24	220	280	140	205
25	230	120	150	161
26	240	150	110	158
27	140	160	100	131
28	260	160	140	180
29	220	220	115	177
30	210	150	110	151
31	160	180	120	151
32	160	170	120	148
33	220	220	240	226
34	160	130	120	136
35	300	200	240	243
36	250	210	200	219
37	210	190	150	182
38	210	200	190	200
39	260	210	140	197
40	250	280	200	241
41	330	240	270	278
42	200	170	170	179
43	200	120	170	160
44	140	130	110	126
45	160	120	80	115
46	100	160	100	117
47	200	80	150	134
48	205	100	130	139
49	130	130	130	130
50	240	180	160	190

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	250	280	140	214
52	360	260	130	230
53	200	140	130	154
54	130	110	110	116
55	190	170	140	165
56	190	80	100	115
57	180	220	160	185
58	290	220	180	226
59	240	230	170	211
60	300	210	150	211
61	180	130	130	145
62	340	290	210	275
63	200	280	170	212
64	225	180	170	190
65	180	150	90	134
66	190	350	300	271
67	120	110	120	117
68	200	170	150	172
69	140	140	140	140
70	310	240	240	261
71	230	180	180	195
72	210	200	180	196
73	270	180	110	175
74	250	220	210	226
75	220	200	200	206
76	220	170	80	144
77	250	160	170	189
78	260	170	220	213
79	260	140	170	184
80	200	200	160	186
81	200	140	100	141
82	300	140	220	210
83	340	290	220	279
84	280	200	160	208
85	150	130	80	116
86	190	160	120	154
87	350	240	140	227
88	210	260	160	206
89	310	250	140	221
90	180	190	80	140
91	200	230	130	182
92	230	200	140	186
93	310	220	120	202
94	380	260	180	261
95	180	170	150	166
96	250	240	180	221
97	210	230	170	202
98	220	210	170	199
99	200	220	140	183
100	240	180	150	186

Boulder Size Survey

Target Area	BD07		Date	2024/1/26
			Person in charge	Arif, Hefryan

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	220	180	130	173
2	180	110	100	126
3	160	80	150	124
4	210	170	90	148
5	180	200	80	142
6	150	120	60	103
7	180	120	70	115
8	190	120	70	117
9	220	140	70	129
10	150	110	90	114
11	160	110	100	121
12	130	120	120	123
13	200	90	80	113
14	170	80	100	111
15	100	90	90	93
16	180	90	80	109
17	190	110	150	146
18	150	90	110	114
19	210	90	90	119
20	140	110	70	103
21	180	110	90	121
22	220	210	100	167
23	160	100	90	113
24	220	120	170	165
25	200	110	100	130
26	170	110	100	123
27	200	140	70	125
28	340	190	200	235
29	200	110	60	110
30	190	80	100	115
31	140	100	80	104
32	160	100	70	104
33	210	160	80	139
34	160	90	70	100
35	160	110	90	117
36	210	120	90	131
37	170	100	60	101
38	230	80	80	114
39	170	130	70	116
40	160	130	110	132
41	160	140	110	135
42	180	100	100	122
43	170	100	90	115
44	210	140	90	138
45	200	190	80	145
46	170	140	90	129
47	220	140	140	163
48	200	130	100	138
49	160	190	90	140
50	150	140	60	108

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	170	90	80	107
52	160	130	90	123
53	210	130	70	124
54	150	120	90	117
55	190	130	70	120
56	200	140	90	136
57	120	120	70	100
58	240	190	140	186
59	180	90	80	109
60	120	110	110	113
61	140	110	70	103
62	250	180	150	189
63	200	100	80	117
64	130	100	70	97
65	110	90	60	84
66	140	90	80	100
67	190	140	80	129
68	170	150	90	132
69	150	150	90	127
70	150	100	90	111
71	170	90	80	107
72	170	110	80	114
73	200	100	80	117
74	170	100	100	119
75	130	120	70	103
76	180	180	150	169
77	270	180	120	180
78	270	190	140	193
79	170	130	120	138
80	200	100	90	122
81	120	140	80	110
82	180	100	70	108
83	160	100	90	113
84	140	90	80	100
85	130	100	70	97
86	120	110	60	93
87	170	120	110	131
88	140	120	80	110
89	180	120	100	129
90	150	110	70	105
91	140	110	60	97
92	200	160	90	142
93	210	130	80	130
94	270	190	150	197
95	220	130	120	151
96	240	180	110	168
97	200	140	130	154
98	190	180	150	172
99	380	250	210	271
100	230	140	90	143

Boulder Size Survey

Target Area BD08

Date 2024/1/25

Person in charge Furuichi, Tono, Dila

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	190	90	120	127
2	260	120	190	181
3	160	120	80	115
4	300	220	150	215
5	250	190	130	183
6	210	200	120	171
7	200	170	160	176
8	230	210	120	180
9	240	130	160	171
10	250	140	170	181
11	170	120	150	145
12	330	140	230	220
13	280	180	180	209
14	190	150	90	137
15	240	200	180	205
16	230	160	150	177
17	200	160	150	169
18	210	160	110	155
19	230	160	150	177
20	240	180	120	173
21	250	190	160	197
22	200	150	140	161
23	190	100	90	120
24	220	200	190	203
25	210	130	240	187
26	240	140	120	159
27	200	180	140	171
28	170	120	110	131
29	260	240	140	206
30	220	120	130	151
31	250	180	190	204
32	220	200	150	188
33	200	160	140	165
34				0
35	200	120	140	150
36	180	160	120	151
37	200	130	130	150
38	200	130	140	154
39	170	150	100	137
40	240	200	160	197
41	170	140	140	149
42	230	150	150	173
43	200	140	140	158
44	400	210	190	252
45	230	200	150	190
46	240	150	140	171
47	270	240	200	235
48	250	160	160	186
49	220	200	160	192
50	260	190	200	215

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	240	200	200	213
52	280	220	200	231
53	300	260	200	250
54	280	190	170	208
55	270	200	160	205
56	250	220	200	222
57	250	160	150	182
58	330	200	200	236
59	160	150	140	150
60	260	200	240	232
61	250	190	150	192
62	300	230	250	258
63	200	140	130	154
64	200	160	160	172
65	200	160	150	169
66	240	200	190	209
67	210	130	140	156
68	190	130	130	148
69	110	90	100	100
70	160	150	120	142
71	220	160	160	178
72	160	120	120	132
73	200	140	120	150
74	190	160	160	169
75	120	110	90	106
76	160	120	140	139
77	190	140	140	155
78	190	170	170	176
79	170	120	120	135
80	250	200	150	196
81	150	140	140	143
82	190	180	140	169
83	190	150	150	162
84	150	130	130	136
85	160	120	110	128
86	180	110	150	144
87	160	140	140	146
88	170	130	140	146
89	170	170	160	167
90	110	90	80	93
91	160	140	140	146
92	140	140	100	125
93	150	140	100	128
94	180	120	100	129
95	220	170	130	169
96	130	120	80	108
97	150	140	10	59
98	190	160	150	166
99	180	140	110	140
100	160	70	210	133

Boulder Size Survey

Target Area	BD09

Date	2024/1/25
Person in charge	Furuichi, Tono, Dila

No	a(cm)	b(cm)	c(cm)	abc^1/3
1	220	170	170	185
2	200	170	180	183
3	310	200	190	228
4	240	240	200	226
5	290	270	200	250
6	220	150	150	170
7	190	140	140	155
8	210	160	150	171
9	220	160	150	174
10	260	230	200	229
11	180	140	140	152
12	240	220	200	219
13	210	160	170	179
14	260	150	160	184
15	220	200	150	188
16	160	160	150	157
17	170	130	130	142
18	190	150	130	155
19	200	190	160	183
20	230	160	160	181
21	280	170	160	197
22	270	170	110	172
23	270	190	180	210
24	260	220	200	225
25	270	220	130	198
26	240	160	130	171
27	210	160	140	168
28	250	200	140	191
29	250	200	140	191
30	220	150	150	170
31	280	200	160	208
32	150	130	120	133
33	230	200	190	206
34	240	160	170	187
35	90	80	60	76
36	200	140	120	150
37	280	280	190	246
38	140	120	120	126
39	180	150	130	152
40	120	90	90	99
41	130	110	100	113
42	140	100	100	112
43	110	100	90	100
44	130	80	100	101
45	120	90	80	95
46	120	90	80	95
47	150	90	90	107
48	150	90	100	111
49	160	140	130	143
50	150	130	120	133

No	a(cm)	b(cm)	c(cm)	abc^1/3
51	160	140	100	131
52	170	160	150	160
53	150	120	100	122
54	160	140	130	143
55	130	100	100	109
56	150	130	130	136
57	170	150	150	156
58	270	200	210	225
59	200	190	160	183
60	270	200	150	201
61	160	140	120	139
62	200	140	130	154
63	200	160	160	172
64	190	140	140	155
65	250	110	110	145
66	200	140	130	154
67	170	130	130	142
68	170	120	120	135
69	190	150	150	162
70	160	140	130	143
71	170	110	110	127
72	170	100	160	140
73	210	190	180	193
74	160	130	140	143
75	230	190	180	199
76	170	120	120	135
77	160	130	130	139
78	150	130	130	136
79	170	120	110	131
80	210	120	150	156
81	210	150	150	168
82	130	130	120	127
83	190	150	150	162
84	240	160	170	187
85	210	200	120	171
86	200	160	100	147
87	190	150	140	159
88	190	130	110	140
89	170	110	110	127
90	200	140	190	175
91	120	110	110	113
92	160	150	140	150
93	160	150	150	153
94	190	140	180	169
95	190	130	150	155
96	170	170	150	163
97	140	130	110	126
98	200	140	110	145
99	140	140	100	125
100	140	100	130	122
















26 Januari 2024 10.01 AM


26 Januari 2024 9.23 AM


26 Januari 2024 10.37 AM


24 Januari 2024 10.08 AM


26 Januari 2024 9.25 AM


26 Januari 2024 10.34 AM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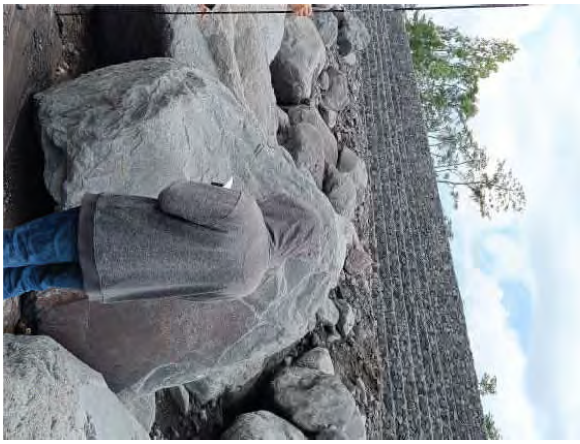


Appendix 4-3-1 Result of Preliminary Detailed Design(Draft)

(Stability Calculation)

CD Kobokan 5

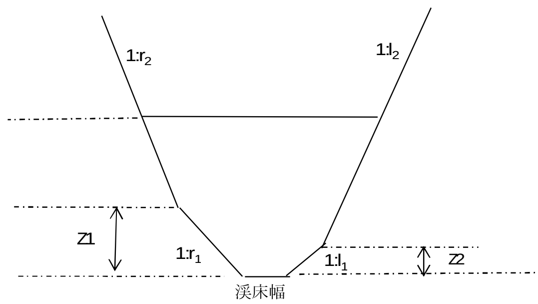
Stability calculation of S2-1 CD Kobokan 5
(Dam height H＜15m)

1. Design specifications

1.1 Design condition list

①Topography, geology, facility shape conditions

Symbol	Item	Unit	Value
	Topographic condition		
	Width of riverbed	m	100.0
$1:r_1$	Flow section of right bank slope (bottom)		1.0
$1:r_2$	Flow section of right bank slope (top)		1.0
$1:l_1$	Flow section of left bank slope (bottom)		1.0
$1:l_2$	Flow section of left bank slope (top)		1.0
θ	Riverbed surface slope		1/17.5
	Flow rate conditions		
A	Target basin area	km^2	19.60
P_{24}	24-hour(day) rainfall	mm	251.0
	Facility shape conditions		
H	Dam height	m	14.5
B	Crest width	m	10.0
b1	Road width	m	6.0
b2	Wing crest width	m	4.0
m	Upstream slope(overflow section)		0.30
m	Upstream slope(non-overflow section)		0.20
n	Downstream slope		0.20
B1	Base width of spillway	m	135.0
	Wing edge		0.5
	Sediment conditions		
d_{95}	Maximum boulder size	m	2.5
	Ground conditions		
τ_0	Shear strength		0
f	Friction coefficient		0.60
qu	Allowable bearing capacity	kN/m^2	600
n	Safety factor against sliding		1.2



②Design constant

We	Apparent unit weight of concrete	kN/m^3	20
We	Unit weight of concrete	kN/m^3	22.6
Wo	Unit weight of water	kN/m^3	11.77
σ	Gravel density	kg/m^3	2,600
ρ	Water density	kg/m^3	1,200
K_{p1}	Coefficient		120
C	Coefficient of overflow		0.6
g	Gravitational acceleration	m/s^2	9.81
C_*	Volumetric concentration of deposited sediment		0.6
Kn	Roughness coefficient(front part)		0.1
C	Coefficient of earth pressure		0.3
ϕ	Internal friction angle of sediment	degree	35.0
τ c	Shear strength of concrete		2,760

Regarding the shear strength, the smaller value in between the dambody and the ground is used in calculation.

1.2 Apparent unit weight of concrete ※1

Symbol	Item	Unit	Value
B1	Base width of spillway	m	135.0
b	Width of opening section	m	3.0
h	Height of opening section	m	6.0
h1	Slab above the opening	m	4.0
	Number of openings	parts	14
W	Dam body volume except opening	m3	16,447
Vc	Dam body volume including opening	m3	18,841
Wc	Unit weight of concrete	kN/m3	22.6
Wγc	Apparent unit weight of concrete	kN/m3	19.73

The apparent unit weight of concrete adopted 20.0 kN/m3

⇒ Adopted value : 20.0 kN/m3

3) If the permeable part is made of concrete, the self-weight of the dam body is calculated using the dam body block volume (V_c) which is calculated by assuming the overflow section as a closed structure, and the dam body block weight (W_{rc}) which is calculated by assuming the overflow section as an open structure (Figure 11). Further, it is noted that the dam body block refers to the concrete block in the permeable part, not the concrete block used at the time of casting.

$$\gamma_{rc} = W_{rc} / V_C \quad \cdots\cdots (6)$$

Where,

γ_{rc} : apparent unit weight of concrete (kN/m^3),

W_{rc} : dam body block weight (kN) calculated by assuming the overflow section as an open structure with concrete,

V_c : dam body block volume (m^3) calculated by assuming the overflow section as a closed structure.

Figure 11 Dam body volume of spillway of the slit part

4) Combinations of design external forces excluding the self-weight of dam body are as shown in Table 4.

※1 Source : Technical Guideline for Designing Sabo Facilities

2. Calculation of flow discharge considering sediment content (1.5Qp)

2.1 Flow discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Target basin area	A	km^2	19.60

The flow discharge considering sediment content is calculated by under rainfall(251mm/24h) with the design exceedance probability.

The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Qp = \frac{1}{3.6} \times K_{f1} \times Pa \times A$$ ※1

Qp : Peak flood discharge (m^3/s)
Pa : Average rainfall intensity during the flood concentration time(mm/hr)
A : Target basin area(km^2)
K_{f1} : Runoff coefficient

Therefore, the peak discharge of water without sediment is as follow.

$$Qp = 315 \quad (m^3/s)$$

Flow discharge considering sediment content is 1.5 times the peak discharge of water without sediment, and calculated as follow.

$$Q = 1.5 \times Qp$$ ※1

$$= 1.5 \times 315$$

$$= 472.5 \quad (m^3/s)$$

※1 Source : Technical Guideline for Establishing Sabo Master Plan

2.2 Calculation of overflow depth for discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Discharge of water considering sediment content	Q	m^3/s	472.50
Coefficient of overflow	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Base width of the spillway	B1	m	135.0
Wing edge slope	m		0.5

The overflow depth for discharge considering sediment content is calculated as follow.

$$Q = \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1 + 2B_2) \times Dh^{3/2}$$ ※1

Where, Dh : Overflow depth (m)

B_2 : Width of overflow (m)

In case of Dh=1.5

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 135.0 + 2 \times 136.5) \times 1.5\hat{}(3/2) \quad 441.37 \quad < 472.50$$

・・・・・NG

In case of Dh=1.6

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 135.0 + 2 \times 136.6) \times 1.6\hat{}(3/2) \quad 486.38 \quad \geqq 472.50$$

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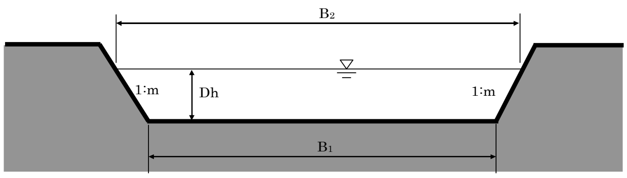
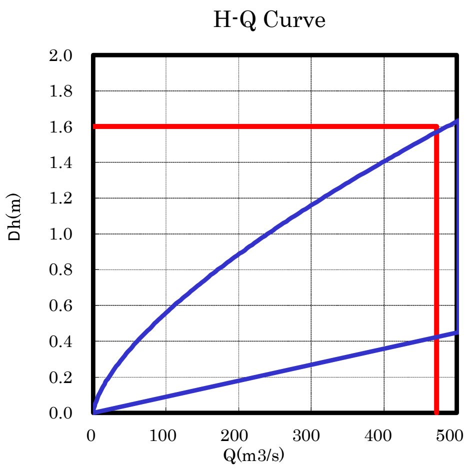
Therefore,

Dh ＝ 1.6 m



Schematic diagram of the cross-sectional shape of spillway



※1 Source : SNI 2851-2021_Desain Sabodam

3. Calculation of debris flow peak discharge (Qsp)

3.1 Debris flow peak discharge (Qsp) ※1

Specifications list

Symbol	Item	Unit	Value
σ	Density of gravel	kg/m^3	2,600
ρ	Density of water	kg/m^3	1,200
Wo	Unit weight of the water	kN/m^3	11.77
ϕ	Internal friction angle of the deposited soil	degrees	35.0
d	Maximum boulder size	m	2.50
$\tan\theta$	Riverbed surface slope		1/17.5
C*	Volumetric concentration of deposited sediment		0.60
Kn	Roughness coefficient (the front part of debris flow)		0.10
B	Width of riverbed	m	100.0

3.1.1 Concentration of debris flow (Takahashi's Equation)

$$Cd = \frac{\rho \cdot \tan\theta}{(\sigma - \rho)\cdot(\tan\phi - \tan\theta)}$$

Where,

ρ	;	Density of water	1,200	(Kg/m^3)	
θ	;	Riverbed surface slope	3.27	(degrees)	(i=1/17.5)
σ	;	Density of gravel	2,600	(Kg/m^3)	
ϕ	;	Internal friction angle of sediment deposited on riverbed	35.0	(degrees)	

$$= \frac{1{,}200 \times \tan 3.27}{(2{,}600 - 1{,}200) \times (\tan 35.0 - \tan 3.27)}$$

$$= 0.08$$

When the calculated value (Cd) is larger than 0.9C*, , Cd is assumed to be equal to 0.9C*. Meanwhile, if the calculated value (Cd) is smaller than 0.3, Cd value is assumed to be 0.3

$\therefore$ Cd = 0.30

3.1.2 Debris flow peak discharge (Qsp)

The peak discharge of debris flow Qsp (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Qp (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp$$

Where,

Cd	:	Concentration of debris flow	=	0.30	
C*	:	Volumetric concentration of deposited sediment	=	0.60	
Qp	:	Peak discharge of water	=	315.00	(m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp = \frac{0.60}{0.60 - 0.30} \times 315 = 630.0$$

Qsp= 630.0 (m^3/sec)

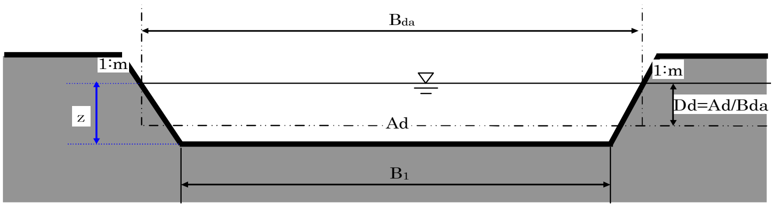
※1 Source : SNI 2851-2021_Desain Sabodam

3.2 Value of the overflow depth for debris flow peak discharge (Qsp).

3.2.1 Calculation of overflow depth ※1

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Debris flow peak discharge	Qsp	m^3/s	630.0
Roughness coefficient	Kn		0.1
Design deposit surface slope	θ	degrees	2.73
Base width of spillway	B1	m	135.0
Wing edge slope	m		0.5



The amount of the debris flow that can flow down through this cross section is determined as follow.

Qsp=U･Ad

Where,

z :	Debris flow surface water level	(m)
U :	Debris flow velocity	(m/s)
Bda :	Width of the flow	(m)
Ad :	Cross-sectional aera of debris flow peak discharge	(m^2)
Dd :	Debris flow depth	(m)

U, Bda, Ad and Dd can be calculated as follow.

U= 1/Kn･$Dd^{2/3}$ ･ $(\sin\theta)^{1/2}$

Dd= Ad/Bda

Ad= (B1+m×z)×z

Bda= B1+2×m×z

If the debris flow depth is 1.57m

Qsp= 2.94 ×213.18

= 626.7 < 630.0 ･････NG

U=	1/0.1	×$1.56^{2/3}$×		$0.05^{1/2}$	=	2.94
Dd=	213.18/		136.57		=	1.56
Ad=	(135.0	+0.5×	1.57)	×1.57	=	213.18
Bda=	135.0	+2×0.50	×1.57		=	136.57

If the debris flow depth is 1.58m

Qsp= 2.95 ×214.55

= 632.9 ≧ 630.0 ･････OK

U=	1/0.1	×$1.57^{2/3}$×		$0.05^{1/2}$	=	2.95
Dd=	214.55/		136.58		=	1.57
Ad=	(135.0	+0.5×	1.58)	×1.58	=	214.55
Bda=	135.0	+2×0.50	×1.58		=	136.58

Therefore, the debris flow depth is adapted as 1.6m

※1 Source : Technical Guideline for Establishing Sabo Master Plan

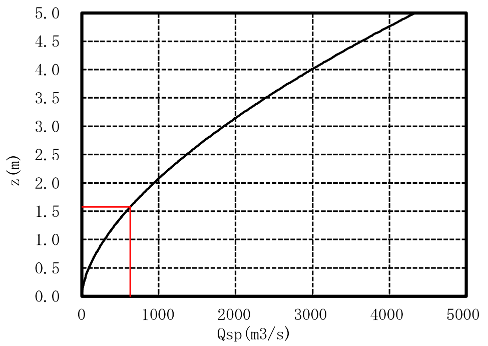
3.2.2 Debris flow specifications over spillway

$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

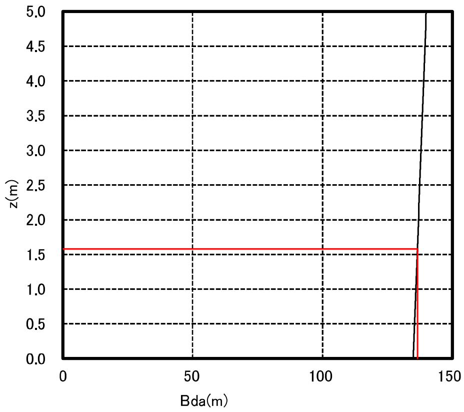
Debris flow discharge (Qsp) 630.0 m^3/s
Design deposit surface slope 1/21.0

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
1.33	135.0	136.32	179.07	1.31	0.1	2.73	2.6	467.3
1.34	135.0	136.33	180.43	1.32	0.1	2.73	2.6	474.5
1.35	135.0	136.34	181.80	1.33	0.1	2.73	2.6	479.9
1.36	135.0	136.35	183.16	1.34	0.1	2.73	2.7	485.3
1.37	135.0	136.36	184.52	1.35	0.1	2.73	2.7	492.6
1.38	135.0	136.38	187.25	1.37	0.1	2.73	2.7	503.7
1.39	135.0	136.39	188.62	1.38	0.1	2.73	2.7	511.1
1.40	135.0	136.40	189.98	1.39	0.1	2.73	2.7	516.7
1.41	135.0	136.41	191.34	1.40	0.1	2.73	2.7	522.3
1.42	135.0	136.42	192.71	1.41	0.1	2.73	2.7	528.0
1.43	135.0	136.43	194.07	1.42	0.1	2.73	2.8	535.6
1.44	135.0	136.44	195.44	1.43	0.1	2.73	2.8	541.3
1.45	135.0	136.45	196.80	1.44	0.1	2.73	2.8	547.1
1.46	135.0	136.46	198.17	1.45	0.1	2.73	2.8	554.8
1.47	135.0	136.47	199.53	1.46	0.1	2.73	2.8	560.6
1.48	135.0	136.48	200.90	1.47	0.1	2.73	2.8	566.5
1.49	135.0	136.49	202.26	1.48	0.1	2.73	2.8	572.3
1.50	135.0	136.50	203.63	1.49	0.1	2.73	2.9	580.3
1.51	135.0	136.51	204.99	1.50	0.1	2.73	2.9	586.2
1.52	135.0	136.52	206.36	1.51	0.1	2.73	2.9	592.2
1.53	135.0	136.53	207.72	1.52	0.1	2.73	2.9	600.3
1.54	135.0	136.54	209.09	1.53	0.1	2.73	2.9	606.3
1.55	135.0	136.55	210.45	1.54	0.1	2.73	2.9	612.4
1.56	135.0	136.56	211.82	1.55	0.1	2.73	2.9	618.5
1.57	135.0	136.57	213.18	1.56	0.1	2.73	2.9	626.7
1.58	135.0	136.58	214.55	1.57	0.1	2.73	3.0	632.9
1.59	135.0	136.59	215.91	1.58	0.1	2.73	3.0	639.0
1.60	135.0	136.60	217.28	1.59	0.1	2.73	3.0	645.3
1.61	135.0	136.61	218.65	1.60	0.1	2.73	3.0	653.7
1.62	135.0	136.62	220.01	1.61	0.1	2.73	3.0	660.0
1.63	135.0	136.63	221.38	1.62	0.1	2.73	3.0	666.3
1.64	135.0	136.64	222.74	1.63	0.1	2.73	3.0	672.6
1.65	135.0	136.65	224.11	1.64	0.1	2.73	3.0	681.2
1.66	135.0	136.66	225.48	1.65	0.1	2.73	3.1	687.7
1.67	135.0	136.67	226.84	1.66	0.1	2.73	3.1	694.1
1.68	135.0	136.68	228.21	1.67	0.1	2.73	3.1	700.6
1.69	135.0	136.69	229.58	1.68	0.1	2.73	3.1	707.1
1.70	135.0	136.70	230.95	1.69	0.1	2.73	3.1	715.9
1.71	135.0	136.71	232.31	1.70	0.1	2.73	3.1	722.4
1.72	135.0	136.72	233.68	1.71	0.1	2.73	3.1	729.0
1.73	135.0	136.73	235.05	1.72	0.1	2.73	3.1	735.7
1.74	135.0	136.74	236.41	1.73	0.1	2.73	3.2	744.6
1.75	135.0	136.75	237.78	1.74	0.1	2.73	3.2	751.3
1.76	135.0	136.76	239.15	1.75	0.1	2.73	3.2	758.1
1.77	135.0	136.77	240.52	1.76	0.1	2.73	3.2	764.8
1.78	135.0	136.78	241.88	1.77	0.1	2.73	3.2	771.5
1.79	135.0	136.79	243.25	1.78	0.1	2.73	3.2	780.8
1.80	135.0	136.80	244.62	1.79	0.1	2.73	3.2	787.6
1.81	135.0	136.81	245.99	1.80	0.1	2.73	3.2	794.5
1.82	135.0	136.82	247.36	1.81	0.1	2.73	3.2	801.4
1.83	135.0	136.83	248.72	1.82	0.1	2.73	3.3	808.3

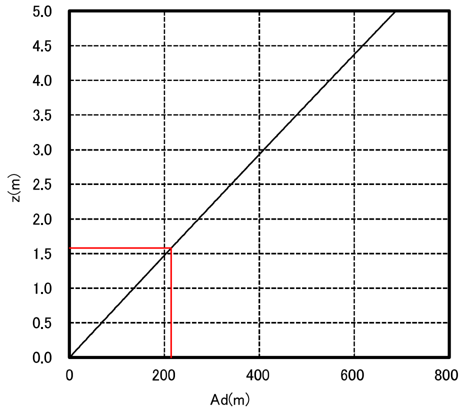
1.58	135.0	136.58	214.55	1.57	0.1	2.73	3.0	632.9



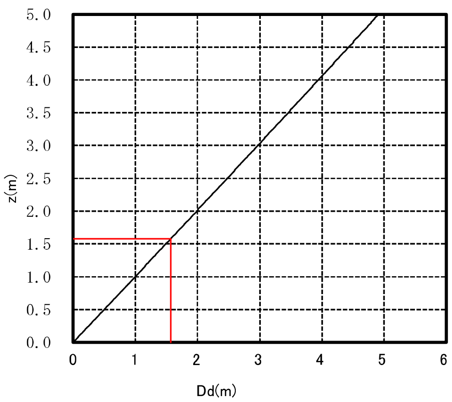
Q_{sp}-Z Curve



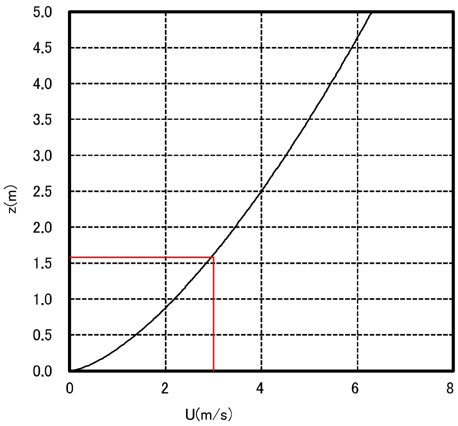
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

4. Design water depth※1

The design water depth is the largest of the values from the calculation results of Table 1.
Therefore, since the Maximum boulder size has the largest value, the design water depth is determined to be 2.5m.

Table 1 Overflow water depth calculated by each method

Item	Depth(m)
Value of the overflow depth of debris flow peak discharge	1.6
Value of overflow depth of discharge considering sediment content	1.6
Maximum boulder size	2.5

The freeboard is set based on Table 2. However, the freeboard height shall be designed according to the riverbed slope and freeboard height ratio to the design water depth must not be less than the value presented in Table 3. The riverbed slope referred here is the design sediment deposition slope.

Table 2 Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6 m
200 ～ 500 m^3/s	0.8 m
500 m^3/s or more	1.0 m

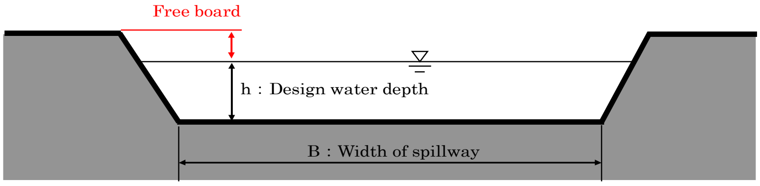


Table 3 Minimum value of the ratio of the freeboard height to the design water depth by riverbed slope

Riverbed slope	(Freeboard height)/(Design water depth)
1/10 or more	0.50
1/10～1/30	0.40
1/30～1/50	0.30
1/50～1/70	0.25

Design deposit surface slope : 1/21
Lowest value of (Freeboard height)/(Design water depth) : 0.4

Since the design discharge is 472.5m3/s, the freeboard is 0.8 m
(Freeboard height)/(Design water depth) is 0.8m / 2.5m = 0.32 < 0.4
(Minimum required freeboard height = 1m)

Therefore, the freeboard height adopts 1.0m

As a result of the above, Spillway height is 2.5 + 1 = 3.5 m

※1 Source : SNI 2851-2021_Desain Sabodam

5. Specifications of debris flow

5.1 Setting of debris flow velocity and depth based on debris flow peak discharge

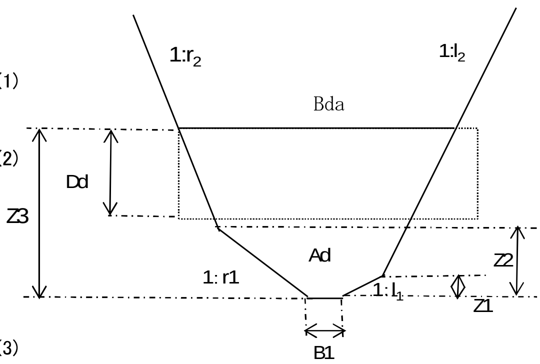
5.1.1 Debris flow cross-section

n_{r1}	:	Flow section right bank slope (bottom)	1.00
n_{r2}	:	Flow section right bank slope (top)	1.00
n_{l1}	:	Flow section left bank slope (bottom)	1.00
n_{l2}	:	Flow section left bank slope (top)	1.00
z	:	Debris flow surface water level	
Bda	:	Width of the flow	

Bda= $n_{r1} \times z + n_{l1} \times z$ (0<z<z1)

$n_{r1} \times z + n_{l2} \times z$ (z1<z<z2)

$n_{r2} \times z + n_{l2} \times z$ (z2<z) ･･･(1)



5.1.2 debris flow depth (Dd)

Dd= Ad ／ Bda ･･･(2)

Ad : Cross-sectional aera of debris flow peak discharge

Ad= Ad1 (0<z<z1)

Ad2 (z1<z<z2)

Ad3 (z2<z)

5.1.3 Debris flow velocity (U)

$$U = \frac{1}{Kn} \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2} \quad \cdots(3)$$

Kn	:	Roughness coefficient	0.10		
θ	:	Riverbed surface slope	3.27	(degrees)	(i=1／17.5)

5.1.4 Calculate debris flow velocity and depth

$Q_{special}$ = U ・ Ad ･･･(4)

From formulas (1), (2), (3), and (4),z is as follows, when Qspecial matches Qsp.

z= 1.79m

From the result of z above,debris flow velocity (U) and depth (Dd) are as follows.

Dd= 1.76m , U= 3.48m/s

5.2 Unit weight of debris flow (γd)

$\gamma d = (\sigma \times Cd) + \{\rho \times (1 - Cd)\}$

$= \{(2{,}600 \times 0.30) + (1{,}200 \times (1- 0.30)\} \times 9.81/1000$

$=$ 15.89 (kN/m^3)

5.3 Drag force of debris flow (F)

$$F = \alpha \cdot \frac{\gamma_d}{g} \cdot Dd \cdot U^2$$

$$= 1.0 \times \frac{15.89}{9.81} \times 1.76 \times 3.48^2$$

$=$ 34.53 (KN／m)

5.4 Unit weight of debris flow in water (ρ di)

ρ di= γ d−Wo

= 15.89 − 11.77 = 4.12 (kN/m3)

5.5 Unit weight of sediment in water (γ s)

γ s=C*($\sigma-\rho$)×g/1000

= 0.6 ×(2,600 − 1,200)×9.81/1000 = 8.24 (kN/m3)

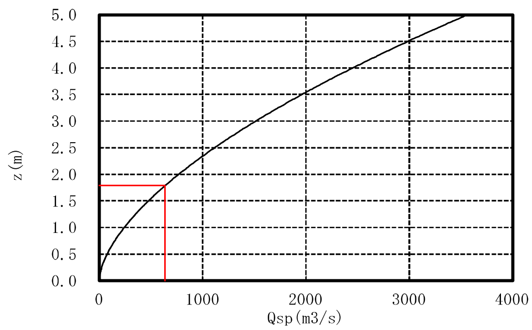
5.6 Debris flow specifications over Riverbed

Debris flow discharge (Qsp) 630.0 m^3/s
Riverbed surface slope 1/17.5

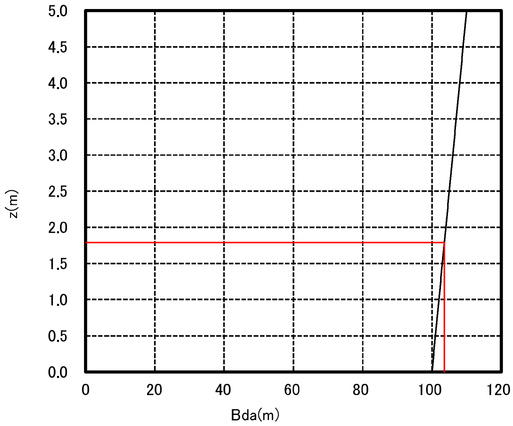
$$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$$

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
1.54	100.0	103.10	156.37	1.52	0.1	3.27	3.16	494.1
1.55	100.0	103.10	157.40	1.53	0.1	3.27	3.17	498.9
1.56	100.0	103.10	158.43	1.54	0.1	3.27	3.18	503.8
1.57	100.0	103.10	159.46	1.55	0.1	3.27	3.20	510.2
1.58	100.0	103.20	160.50	1.56	0.1	3.27	3.21	515.2
1.59	100.0	103.20	161.53	1.57	0.1	3.27	3.23	521.7
1.60	100.0	103.20	162.56	1.58	0.1	3.27	3.24	526.6
1.61	100.0	103.20	163.59	1.58	0.1	3.27	3.24	530.0
1.62	100.0	103.20	164.62	1.59	0.1	3.27	3.25	535.0
1.63	100.0	103.30	165.66	1.60	0.1	3.27	3.27	541.7
1.64	100.0	103.30	166.69	1.61	0.1	3.27	3.28	546.7
1.65	100.0	103.30	167.72	1.62	0.1	3.27	3.29	551.7
1.66	100.0	103.30	168.76	1.63	0.1	3.27	3.31	558.5
1.67	100.0	103.30	169.79	1.64	0.1	3.27	3.32	563.7
1.68	100.0	103.40	170.82	1.65	0.1	3.27	3.33	568.8
1.69	100.0	103.40	171.86	1.66	0.1	3.27	3.35	575.7
1.70	100.0	103.40	172.89	1.67	0.1	3.27	3.36	580.9
1.71	100.0	103.40	173.92	1.68	0.1	3.27	3.38	587.8
1.72	100.0	103.40	174.96	1.69	0.1	3.27	3.39	593.1
1.73	100.0	103.50	175.99	1.70	0.1	3.27	3.40	598.3
1.74	100.0	103.50	177.03	1.71	0.1	3.27	3.42	605.4
1.75	100.0	103.50	178.06	1.72	0.1	3.27	3.43	610.7
1.76	100.0	103.50	179.10	1.73	0.1	3.27	3.44	616.1
1.77	100.0	103.50	180.13	1.74	0.1	3.27	3.46	623.2
1.78	100.0	103.60	181.17	1.75	0.1	3.27	3.47	628.6
1.79	100.0	103.60	182.20	1.76	0.1	3.27	3.48	634.0
1.80	100.0	103.60	183.24	1.77	0.1	3.27	3.49	639.5
1.81	100.0	103.60	184.28	1.78	0.1	3.27	3.51	646.8
1.82	100.0	103.60	185.31	1.79	0.1	3.27	3.52	652.2
1.83	100.0	103.70	186.35	1.80	0.1	3.27	3.53	657.8
1.84	100.0	103.70	187.39	1.81	0.1	3.27	3.55	665.2
1.85	100.0	103.70	188.42	1.82	0.1	3.27	3.56	670.7
1.86	100.0	103.70	189.46	1.83	0.1	3.27	3.57	676.3
1.87	100.0	103.70	190.50	1.84	0.1	3.27	3.59	683.8
1.88	100.0	103.80	191.53	1.85	0.1	3.27	3.60	689.5
1.89	100.0	103.80	192.57	1.86	0.1	3.27	3.61	695.1
1.90	100.0	103.80	193.61	1.87	0.1	3.27	3.63	702.8
1.91	100.0	103.80	194.65	1.87	0.1	3.27	3.63	706.5
1.92	100.0	103.80	195.69	1.88	0.1	3.27	3.64	712.3
1.93	100.0	103.90	196.72	1.89	0.1	3.27	3.65	718.0
1.94	100.0	103.90	197.76	1.90	0.1	3.27	3.66	723.8
1.95	100.0	103.90	198.80	1.91	0.1	3.27	3.68	731.5
1.96	100.0	103.90	199.84	1.92	0.1	3.27	3.69	737.4
1.97	100.0	103.90	200.88	1.93	0.1	3.27	3.70	743.2
1.98	100.0	104.00	201.92	1.94	0.1	3.27	3.72	751.1
1.99	100.0	104.00	202.96	1.95	0.1	3.27	3.73	757.0
2.00	100.0	104.00	204.00	1.96	0.1	3.27	3.74	762.9
2.01	100.0	104.00	205.04	1.97	0.1	3.27	3.75	768.9
2.02	100.0	104.00	206.08	1.98	0.1	3.27	3.77	776.9
2.03	100.0	104.10	207.12	1.99	0.1	3.27	3.78	782.9
2.04	100.0	104.10	208.16	2.00	0.1	3.27	3.79	788.9

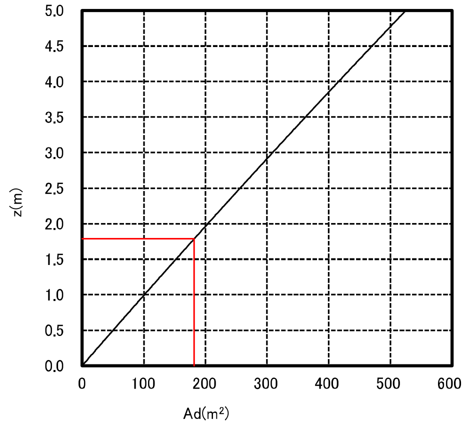
1.79	100.0	103.58	182.20	1.76	0.1	3.27	3.48	634.0



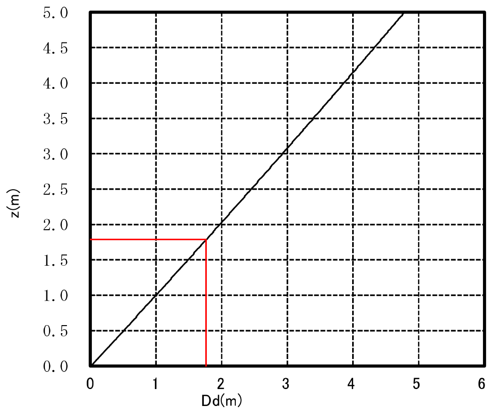
Q_{sp}-Z Curve



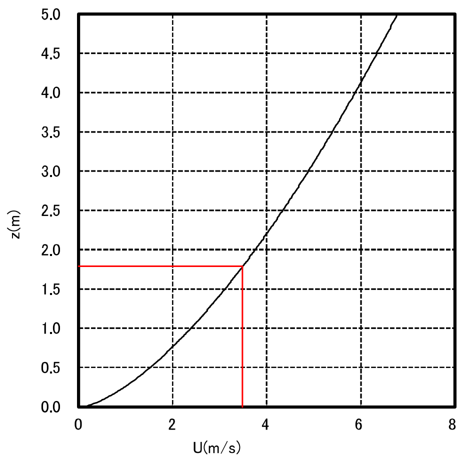
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

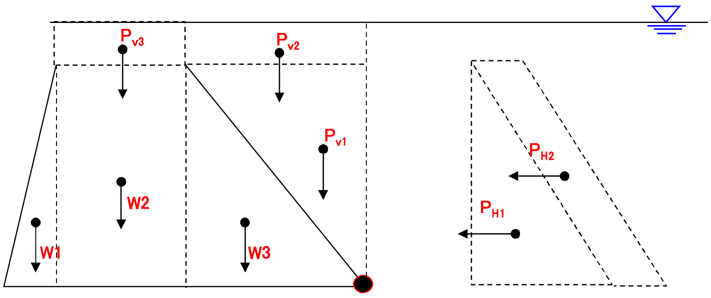
6. Stability analysis

6.1 Result of Stability analysis (overflow section)

During flooding					
Stability against overturning	e	2.191	≦	2.208	OK
Stability against sliding	n	1.25	≧	1.2	OK
Maximum ground reaction force	σ max	521.3	≦	600	OK
Minimum ground reaction force	σ min	2.1	≧	0	OK
During debris flow					
Stability against overturning	e	1.472	≦	2.208	OK
Stability against sliding	n	1.37	≧	1.2	OK
Maximum ground reaction force	σ max	426.8	≦	600	OK
Minimum ground reaction force	σ min	85.4	≧	0	OK

6.2 Verification of stability of main body (overflow section)

6.2.1 During flooding



Schematic diagram of design external forces

Remarks：Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 2.90 × 14.50 × 20.00 13.25 − 2/3 × 2.90	420.500		11.317	4,758.658
W2	6.00 × 14.50 × 20.00 4.35 ＋ 1/2 × 6.00	1,740.000		7.350	12,789.000
W3	1/2 × 4.35 × 14.50 × 20.00 2/3 × 4.35	630.750		2.900	1,829.175
Hydrostatic pressure P_{V1}	1/2 × 4.35 × 14.50 × 11.77 1/3 × 4.35	371.196		1.450	538.235
P_{V2}	4.35 × 2.50 × 11.77 1/2 × 4.35	127.999		2.175	278.397
P_{V3}	6.00 × 2.50 × 11.77 4.35 ＋ 1/2 × 6.00	176.550		7.350	1,297.643
P_{H1}	1/2 × 14.50 × 14.50 × 11.77 1/3 × 14.50		1,237.321	4.833	5,980.386
P_{H2}	2.50 × 14.50 × 11.77 1/2 × 14.50		426.663	7.250	3,093.303
TOTAL		3,466.995	1,663.984		30,564.797

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{30564.797}{3466.995} = 8.816$ m

Eccentric distance
$e = d - B/2 = 8.816 - 13.250 / 2 = 2.191$ m
$e = 2.191$ m $\leqq$ $B/6 = 2.208$ m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 3466.995 + 0.0 \times 13.250}{1663.984} = 1.25$$

$n = 1.25 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{3467.00}{13.25} \times \left(1 - \frac{6 \times 2.19}{13.25}\right) = 2.1 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

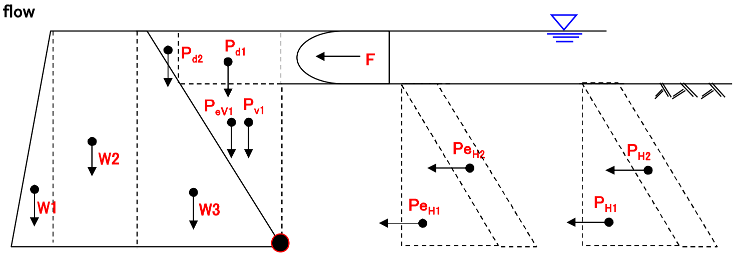
Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{3467.00}{13.25} \times \left(1 + \frac{6 \times 2.19}{13.25}\right) = 521.3 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

OK

6.2.2 During bebris flow



Schematic diagram of design external forces

Remarks：Upper part of the formula is the calculation of force
Lower part of the formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 2.90 × 14.50 × 20.00 13.25 − 2/3 × 2.90	420.500		11.317	4,758.658
W2	6.00 × 14.50 × 20.00 4.35 ＋ 1/2 × 6.00	1,740.000		7.350	12,789.000
W3	1/2 × 4.35 × 14.50 × 20.00 2/3 × 4.35	630.750		2.900	1,829.175
Hydrostatic pressure P_{V1}	1/2 × 3.82 × 12.74 × 11.77 1/3 × 3.82	286.554		1.274	365.070
P_{H1}	1/2 × 12.74 × 12.74 × 11.77 1/3 × 12.74		955.180	4.247	4,056.332
P_{H2}	1.76 × 12.74 × 11.77 1/2 × 12.74		263.912	6.370	1,681.117
Earth pressure P_{ev1}	1/2 × 3.82 × 12.74 × 8.24 1/3 × 3.82	200.612		1.274	255.580
P_{eH1}	1/2 × 0.30 × 12.74^2 × 8.24 1/3 × 12.74		200.612	4.247	851.933
P_{eH2}	0.30 × 1.76 × 12.74 × 4.12 1/2 × 12.74		27.710	6.370	176.513
Mass of debris flow Pd1	3.82 × 1.76 × 15.89 1/2 × 3.82	106.888		1.911	204.263
Pd2	1/2 × 0.53 × 1.76 × 15.89 4.35 − 2/3 × 0.53	7.383		3.998	29.517
Drag force of debris flow F	1.00 × 15.89 × 3.48^2 × 1.76 /9.81 12.74 ＋ 1/2 × 1.76		34.524	13.620	470.222
TOTAL		3,392.687	1,481.938		27,467.380

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{27467.380}{3392.687} = 8.096$ m

Eccentric distance

$e = d - B/2 = 8.096 - 13.250 / 2 = 1.472$ m

$e = 1.472$ m $\leqq B/6 = 2.208$ m .... OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 3392.687 + 0.0 \times 13.25}{1481.938} = 1.37$$

$n = 1.37 \geqq 1.2$ .... OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{3392.69}{13.25} \times \left(1 - \frac{6 \times 1.47}{13.25}\right) = 85.4 \ kN/m^2 \geqq 0 \ kN/m^2$$

.... OK

Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{3392.69}{13.25} \times \left(1 + \frac{6 \times 1.47}{13.25}\right) = 426.8 \ kN/m^2 \leqq 600 \ kN/m^2$$

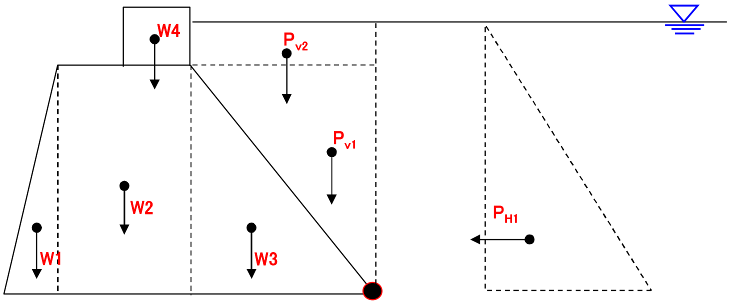
.... OK

6.3 Result of Stability analysis (non-overflow section)

During flooding					
Stability against overturning	e	1.454	≦	2.633	OK
Stability against sliding	n	1.58	≧	1.2	OK
Maximum ground reaction force	σ max	441.5	≦	600	OK
Minimum ground reaction force	σ min	127.4	≧	0	OK
During debris flow					
Stability against overturning	e	1.264	≦	2.633	OK
Stability against sliding	n	1.50	≧	1.2	OK
Maximum ground reaction force	σ max	436.8	≦	600	OK
Minimum ground reaction force	σ min	153.5	≧	0	OK

6.4 Verification of stability of main body (non-overflow section)

6.4.1 During flooding



Schematic diagram of design external forces

Remarks : Upper part of the formula is the calculation of force
Lower part of the formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN·m)
Self-weight of dam W1	1/2 × 2.90 × 14.50 × 20.00 15.80 − 2/3 × 2.90	420.500		13.867	5,830.933
W2	10.00 × 14.50 × 20.00 2.90 + 1/2 × 10.00	2,900.000		7.900	22,910.000
W3	1/2 × 2.90 × 14.50 × 20.00 2/3 × 2.90	420.500		1.933	812.967
W4	3.50 × 6.00 × 20.00 2.90 + 6.00 × 1/2	420.000		5.900	2,478.000
Hydrostatic pressure P_{v1}	1/2 × 2.90 × 14.50 × 11.77 1/3 × 2.90	247.464		0.967	239.215
P_{v2}	2.90 × 2.50 × 11.77 1/2 × 2.90	85.333		1.450	123.732
P_{H1}	1/2 × 17.00 × 17.00 × 11.77 1/3 × 17.00		1,700.765	5.667	9,637.668
TOTAL		4,493.797	1,700.765		42,032.515

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{42032.515}{4493.797} = 9.353$ m

Eccentric distance

$e = d - B/2 = 9.353 - 15.800 / 2 = 1.454$ m

$e = 1.454$ m $\leqq B/6 = 2.633$ m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 4493.797 + 0.0 \times 15.800}{1700.765} = 1.58$$

$n = 1.58 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{4493.80}{15.80} \times \left(1 - \frac{6 \times 1.45}{15.80}\right) = 127.4 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

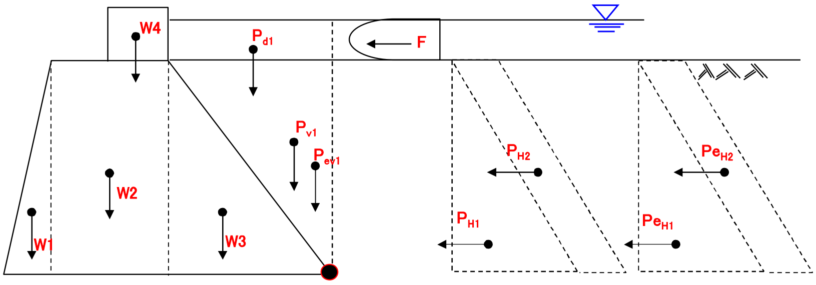
Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{4493.80}{15.80} \times \left(1 + \frac{6 \times 1.45}{15.80}\right) = 441.5 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

OK

6.4.2 During bebris flow



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 2.90 × 14.50 × 20.00 15.80 − 2/3 × 2.90	420.500		13.867	5,830.933
W2	10.00 × 14.50 × 20.00 2.90 + 1/2 × 10.00	2,900.000		7.900	22,910.000
W3	1/2 × 2.90 × 14.50 × 20.00 2/3 × 2.90	420.500		1.933	812.967
W4	3.50 × 6.00 × 20.00 2.90 + 6.00 × 1/2	420.000		5.900	2,478.000
Hydrostatic pressure P_{V1}	1/2 × 2.90 × 14.50 × 11.77 1/3 × 2.90	247.464		0.967	239.215
P_{H1}	1/2 × 14.50 × 14.50 × 11.77 1/3 × 14.50		1,237.321	4.833	5,980.386
P_{H2}	1.76 × 14.50 × 11.77 1/2 × 14.50		300.370	7.250	2,177.685
Earth pressure P_{ev1}	1/2 × 2.90 × 14.50 × 8.24 1/3 × 2.90	173.246		0.967	167.471
P_{eH1}	1/2 × 0.30 × 14.50^2 × 8.24 1/3 × 14.50		259.869	4.833	1,256.034
P_{eH2}	0.30 × 1.76 × 14.50 × 4.12 1/2 × 14.50		31.540	7.250	228.665
Mass of debris flow Pd1	2.90 × 1.76 × 15.89 1/2 × 2.90	81.103		1.450	117.599
Drag force of debris flow F	1.00 × 15.89 × 3.48^2 × 1.76 /9.81 14.50 + 1/2 × 1.76		34.524	15.380	530.985
TOTAL		4,662.813	1,863.625		42,729.940

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{42729.940}{4662.813} = 9.164$ m

Eccentric distance

$e = d - B/2 = 9.164 - 15.800 / 2 = 1.264$ m

$e = 1.264$ m $\leqq B/6 = 2.633$ m　OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 4662.813 + 0.0 \times 15.800}{1863.625} = 1.50$$

$n = 1.50 \geqq 1.2$　OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{4662.81}{15.80} \times \left(1 - \frac{6 \times 1.26}{15.80}\right) = 153.5 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{4662.81}{15.80} \times \left(1 + \frac{6 \times 1.26}{15.80}\right) = 436.8 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

OK

7. Downstream riverbed protection works※1

7.1 Thickness of the apron

Thickness of the apron is calculated as follow.
Empirical formula (If there is the water cushion)

$t = 0.1 \times (0.6H_1 + 3h_3 - 1.0)$

Where,
- t : Thickness of the apron (m)
- H_1 : Height from the top of the apron to the base of the spillway (m)
- h_3 : Design water depth(m)

If H : Dam height(m)
Then $H_1 = H-t$

$t = 0.1 \times (0.6(H-t) + 3h_3 - 1.0)$

$= -0.06t + (0.1 \times (0.6H + 3h_3 - 1.0)$

Therefore,

$t = 0.1 \times (0.6H + 3h_3 - 1.0) / 1.06$

In case of the planned main dam,
Then $H =$ 14.5 m
$h_3 =$ 2.5 m

$t = 0.1 \times (0.6 \times 14.5 + 3 \times 2.5 - 1.0) / 1.06$
$=$ 1.43396 m

$t =$ 1.5 m

⇒ Adopted value : 2.0 m

7.2 Overlap height

Overlap height between main-dam and sub-dam is calculated as follow.
Empirical formula
$H_2 = (1/3 \sim 1/4)\ H$

Where,
- H : Dam height(m)
- H_2 : Overlap height(m)

In addition, there are many cases where 1/3 to 1/4 is used as follow.
If the foundation is rock : 1/4
If the foundation is gravel : 1/3

H : Dam height (m)	Foundation	Coefficient	H_2 : Overlap height	Adoption
14.5	Gravel	1/3	4.83	4.9

⇒ Adopted value : 5.0 m

7.3 Length of the apron

Length of the apron (L) is calculated as follow.

$L = (1.5 \sim 2.0) \times (H_1 + H_3)$

$L = 1.5 \times (H_1 + H_3) =$ 22.5 m
$L = 2.0 \times (H_1 + H_3) =$ 30.0 m

⇒ Adopted value : 30.0 m

※1 Source : SNI 2851-2021_Desain Sabodam

8. Ability to flow down of the opening section

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Number of opening section		Parts	14
Peak discharge of water	Q	m^3/s	315
Target discharge amount per part		$m^3/s/part$	22.50
Coefficient	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Width of opening section	B1	m	3.0

The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Q = \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1 + 2B_2) \times Dh^{3/2}$$

Where, Dh : Water depth (m)

B_2 : Width of overflow (m)

In case of Dh=2.6

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 3.0+2 \times 3.0) \times 2.6\hat{}(3/2) = 22.28 \quad < 22.50$$

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In case of Dh=2.7

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 3.0+2 \times 3.0) \times 2.7\hat{}(3/2) = 23.58 \quad \geqq 22.50$$

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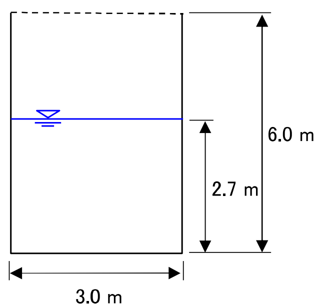
Therefore,

Dh ＝ 2.7 m



Schematic diagram of opening section

The overflow water depth is 2.7m, and the opening height is 6m, so the target water flow can flow down only through the opening section.

CD Pelintas Curah Lengkong2

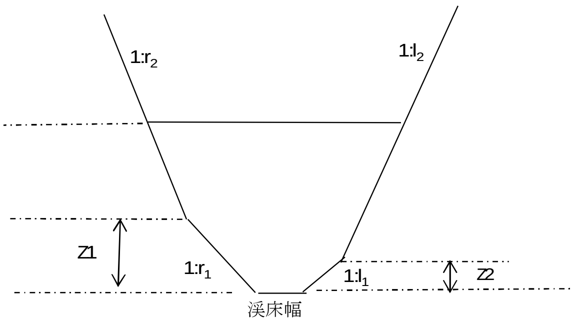
Stability calculation of S2-2CD Pelintas Curah Lengkong2
(Dam height H＜15m)

1. Design specifications

1. 1 Design condition list

①Topography, geology, facility shape conditions

Symbol	Item	Unit	Value
	Topographic condition		
	Width of Riverbed	m	100.0
$1:r_1$	Flow section right bank slope (bottom)		1.0
$1:r_2$	Flow section right bank slope (top)		1.0
$1:l_1$	Flow section left bank slope (bottom)		1.0
$1:l_2$	Flow section left bank slope (top)		1.0
θ	Riverbed surface slope		1/16
	Flow rate conditions		
A	Target basin area	km^2	19.60
P_{24}	24-hour(day) rainfall	mm	251.0
	Facility shape conditions		
H	Dam height	m	11.5
B	Crest width	m	10.0
b1	Road width	m	6.0
b2	Wing crest width	m	4.0
m	Upstream slope(overflow section)		0.30
m	Upstream slope(non-overflow section)		0.20
n	Downstream slope		0.20
B1	Base width of spillway	m	45.0
	Wing edge		0.5
	Sediment conditions		
d_{95}	Maximum boulder size	m	2.5
	Ground conditions		
τ_0	Shear strength		600
f	Friction coefficient		0.70
qu	Allowable bearing capacity	kN/m^2	1,200
n	Safety factor against sliding		4



②Design constant

Symbol	Item	Unit	Value
We	Apparent unit weight of concrete	kN/m^3	20
We	Unit weight of concrete	kN/m^3	22.6
Wo	Unit weight of water	kN/m^3	11.77
σ	Gravel density	kg/m^3	2,600
ρ	Water density	kg/m^3	1,200
K_{p1}	Coefficient		120
C	Coefficient of overflow		0.6
g	Gravitational acceleration	m/s^2	9.81
C_*	metric concentration of deposited sediment		0.6
Kn	Roughness coefficient(front part)		0.1
C	Coefficient of earth pressure		0.3
ϕ	Internal friction angle of sediment	degree	35.0
τc	Shear strength of concrete		2,760

Regarding the shear strength, the smaller value in between the dambody and the ground is used in calculation.

1．2 Apparent unit weight of concrete ※1

Symbol	Item	Unit	Value
B1	Base width of spillway	m	45.0
b	Width of opening section	m	3.0
h	Height of opening section	m	4.5
h1	Slab above the opening	m	4.0
	Number of openings	parts	7
W	Dam body volume except opening	m3	3,731
Vc	Dam body volume including opening	m3	4,593
Wc	Unit weight of concrete	kN/m3	22.6
Wγc	Apparent unit weight of concrete	kN/m3	18.36

The apparent unit weight of concrete adopts 20.0 kN/m3

⇒ Adopted value : 20.0 kN/m3

3) If the permeable part is made of concrete, the self-weight of the dam body is calculated using the dam body block volume (V_c) which is calculated by assuming the overflow section as a closed structure, and the dam body block weight (W_{rc}) which is calculated by assuming the overflow section as an open structure (Figure 11). Further, it is noted that the dam body block refers to the concrete block in the permeable part, not the concrete block used at the time of casting.

$$\gamma_{rc} = W_{rc} / V_C \quad \cdots\cdots (6)$$

Where,

γ_{rc} : apparent unit weight of concrete (kN/m³),

W_{rc} : dam body block weight (kN) calculated by assuming the overflow section as an open structure with concrete,

V_c : dam body block volume (m³) calculated by assuming the overflow section as a closed structure.

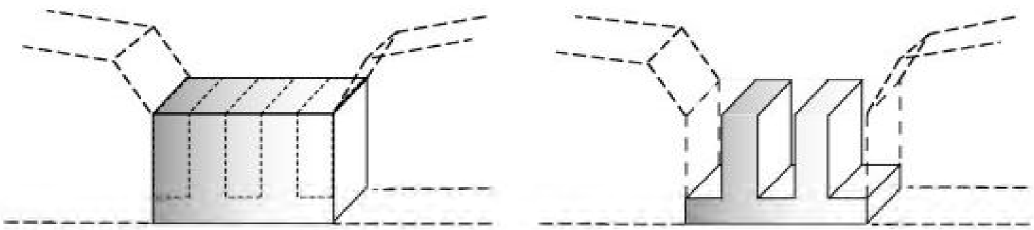


Figure 11 Dam body volume of spillway of the slit part

4) Combinations of design external forces excluding the self-weight of dam body are as shown in Table 4.

※1 Source : Technical Guideline for Designing Sabo Facilities

2. Calculation of flow discharge considering sediment content (1.5Qp)

2. 1 Flow discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Target basin area	A	km^2	19.60

The flow discharge considering sediment content is calculated by under rainfall(251mm/24h) with the design exceedance probability.
The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Qp = \frac{1}{3.6} \times K_{f1} \times Pa \times A$$ ※1

Qp : Peak flood discharge (m^3/s)
Pa : Average rainfall intensity during the flood concentration time(mm/hr)
A : Target basin area(km^2)
K_{f1} : Runoff coefficient

Therefore, the peak discharge of water without sediment is as follow.

$$Qp = 315 \quad (m^3/s)$$

Flow discharge considering sediment content is 1.5 times the peak discharge of water without sediment, and calculated as follow.

$$Q = 1.5 \times Qp$$ ※1
$$= 1.5 \times 315$$
$$= 472.5 \quad (m^3/s)$$

※1 Source : Technical Guideline for Establishing Sabo Master Plan

2.2 Calculation of overflow depth for discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Discharge of water considering sediment content	Q	m^3/s	472.50
Coefficient of overflow	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Base width of the spillway	B1	m	45.0
Wing edge slope	m		0.5

The overflow depth in regards to discharge of water containing fine sediments is calculated as follow.

$$Q = \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1 + 2B_2) \times Dh^{3/2}$$ ※1

Where, Dh : Overflow depth (m)
B_2 : Width of overflow (m)

In case of Dh=3.2

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 45.0 + 2 \times 48.2) \times 3.2\hat{}(3/2) = 469.38 \quad < 472.50$$

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In case of Dh=3.3

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 45.0 + 2 \times 48.3) \times 3.3\hat{}(3/2) = 491.98 \quad \geqq 472.50$$

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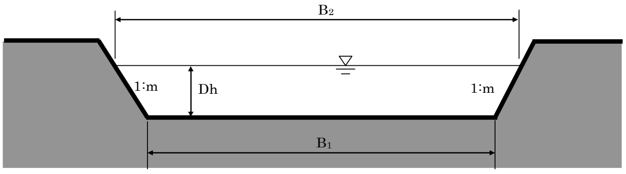
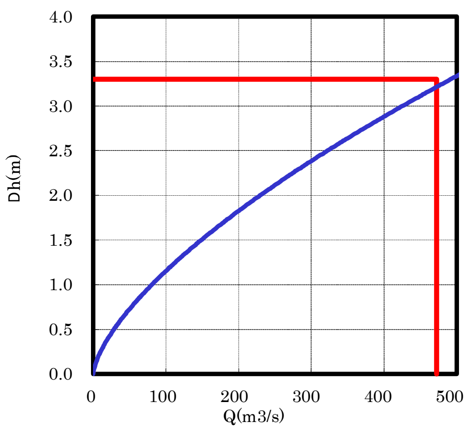
Therefore,
Dh ＝ 3.3 m



Schematic diagram of the cross-sectional shape of Spillway

H-Q Curve



※1 Source : SNI 2851-2021_Desain Saboda

3. Calculation of Peak discharge of debris flow (Qsp)

3. 1 Peak discharge of debris flow (Qsp) ※1

Specifications list

Symbol	Item	Unit	Value
σ	Density of gravel	kg/m^3	2,600
ρ	Density of water	kg/m^3	1,200
Wo	Unit weight of the water	kN/m^3	11.77
ϕ	Internal friction angle of the deposited soil	degrees	35.0
d	Maximum boulder size	m	2.50
$\tan\theta$	Riverbed surface slope		1/16
C_*	Volumetric concentration of deposited sediment		0.60
Kn	Roughness coefficient (the front part of debris flow)		0.10
B	Width of riverbed	m	100.0

3.1.1 Concentration of debris flow (Takahashi's Equation)

$$Cd = \frac{\rho \cdot \tan\theta}{(\sigma - \rho)\cdot(\tan\phi - \tan\theta)}$$

Where,

ρ	;	Density of water	1,200	(Kg/m^3)
θ	;	Riverbed surface slope	3.58	(degrees) (i=1/16)
σ	;	Density of gravel	2,600	(Kg/m^3)
ϕ	;	Internal friction angle of sediment deposited on riverbed	35.0	(degrees)

$$= \frac{1{,}200 \times \tan 3.58}{(2{,}600 - 1{,}200) \times (\tan 35.0 - \tan 3.58)}$$

$$= 0.08$$

When the calculated value (Cd) is larger than 0.9C$_*$, Cd is assumed to be equal to 0.9C$_*$. Meanwhile, if the calculated value (Cd) is smaller than 0.3, Cd value is assumed to be 0.3

$\therefore$ Cd = 0.30

3.1.2 Peak discharge of debris flow (Qsp)

The peak discharge of debris flow Qsp (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Qp (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp$$

Where,

Cd	:	Concentration of debris flow	=	0.30	
C*	:	Volumetric concentration of deposited sediment	=	0.60	
Qp	:	Peak discharge of water	=	315.00	(m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp = \frac{0.60}{0.60 - 0.30} \times 315 = 630.0$$

Qsp= 630.0 (m^3/sec)

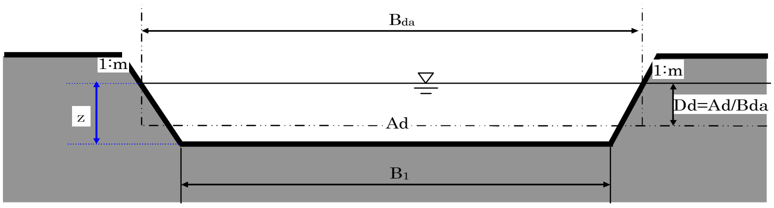
※1 Source : SNI 2851-2021_Desain Sabodam

3. 2 Value of the overflow depth in regards to the debris flow peak discharge (Qsp).

3.2.1 Calculation of overflow depth ※1

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Debris flow peak discharge	Qsp	m^3/s	630.0
Roughness coefficient	Kn		0.1
Design deposit surface slope	θ	degrees	2.98
Base width of spillway	B1	m	45.0
Wing edge slope	m		0.5



The amount of the debris flow that can flow down through this cross section is determined as follow.

Qsp=U･Ad

Where,	z :	Debris flow surface water level	(m)
	U :	Debris flow velocity	(m/s)
	Bda :	Width of the flow	(m)
	Ad :	Cross-sectional aera of debris flow peak discharge	(m^2)
	Dd :	Debris flow depth	(m)

U, Bda, Ad and Dd can be calculated as follow.

U= 1/Kn･$Dd^{2/3}$ ･ $(\sin\theta)^{1/2}$

Dd= Ad/Bda

Ad= (B1+m × z) × z

Bda= B1+2 × m × z

If the debris flow depth is 2.95m

Qsp= 4.59 × 137.10

= 629.2 < 630.0 ･････NG

U=	1/0.1	× 2.86	$^{2/3}$×		0.05 $^{1/2}$	=	4.59
Dd=	137.10/		47.95			=	2.86
Ad=	(45.0	+0.5 ×	2.95)	× 2.95		=	137.10
Bda=	45.0	+2 × 0.50	× 2.95			=	47.95

If the debris flow depth is 2.96m

Qsp= 4.6 × 137.58

= 632.8 ≧ 630.0 ･････OK

U=	1/0.1	× 2.87	$^{2/3}$×		0.05 $^{1/2}$	=	4.6
Dd=	137.58/		47.96			=	2.87
Ad=	(45.0	+0.5 ×	2.96)	× 2.96		=	137.58
Bda=	45.0	+2 × 0.50	× 2.96			=	47.96

Therefore, the debris flow depth is adapted as 3.0m

※1 Source : Technical Guideline for Establishing Sabo Master Plan

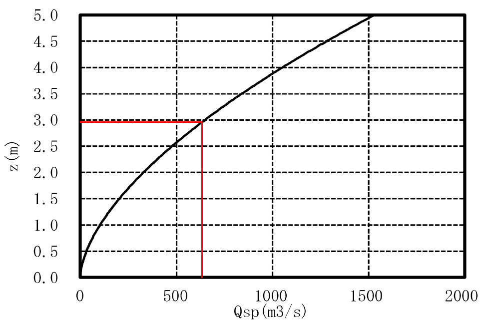
3.2.2 Debris flow specifications over spillway

$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

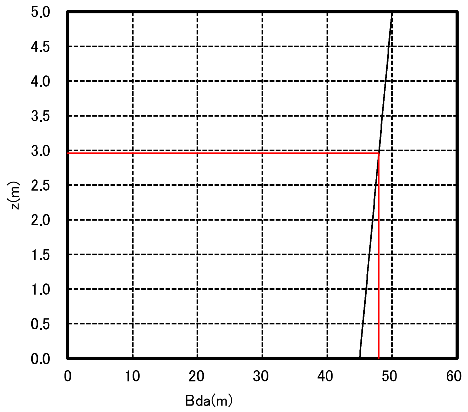
Debris flow discharge (Qsp)	630.0 m^3/s
Design deposit surface slope	1/19.2

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
2.71	45.0	47.71	125.62	2.63	0.1	2.98	4.3	545.1
2.72	45.0	47.72	126.10	2.64	0.1	2.98	4.4	549.7
2.73	45.0	47.73	126.58	2.65	0.1	2.98	4.4	553.1
2.74	45.0	47.74	127.05	2.66	0.1	2.98	4.4	556.4
2.75	45.0	47.75	127.53	2.67	0.1	2.98	4.4	559.8
2.76	45.0	47.76	128.01	2.68	0.1	2.98	4.4	563.2
2.77	45.0	47.77	128.49	2.69	0.1	2.98	4.4	566.6
2.78	45.0	47.78	128.96	2.70	0.1	2.98	4.4	570.0
2.79	45.0	47.79	129.44	2.71	0.1	2.98	4.4	573.4
2.80	45.0	47.80	129.92	2.72	0.1	2.98	4.4	576.8
2.81	45.0	47.81	130.40	2.73	0.1	2.98	4.5	580.2
2.82	45.0	47.82	130.88	2.74	0.1	2.98	4.5	583.7
2.83	45.0	47.83	131.35	2.75	0.1	2.98	4.5	588.4
2.84	45.0	47.84	131.83	2.76	0.1	2.98	4.5	591.9
2.85	45.0	47.85	132.31	2.77	0.1	2.98	4.5	595.3
2.86	45.0	47.86	132.79	2.77	0.1	2.98	4.5	597.5
2.87	45.0	47.87	133.27	2.78	0.1	2.98	4.5	601.0
2.88	45.0	47.88	133.75	2.79	0.1	2.98	4.5	604.5
2.89	45.0	47.89	134.23	2.80	0.1	2.98	4.5	608.0
2.90	45.0	47.90	134.71	2.81	0.1	2.98	4.5	611.5
2.91	45.0	47.91	135.18	2.82	0.1	2.98	4.6	615.0
2.92	45.0	47.92	135.66	2.83	0.1	2.98	4.6	618.6
2.93	45.0	47.93	136.14	2.84	0.1	2.98	4.6	622.1
2.94	45.0	47.94	136.62	2.85	0.1	2.98	4.6	625.7
2.95	45.0	47.95	137.10	2.86	0.1	2.98	4.6	629.2
2.96	45.0	47.96	137.58	2.87	0.1	2.98	4.6	632.8
2.97	45.0	47.96	137.58	2.87	0.1	2.98	4.6	632.8
2.98	45.0	47.97	138.06	2.88	0.1	2.98	4.6	637.8
2.99	45.0	47.98	138.54	2.89	0.1	2.98	4.6	641.4
3.00	45.0	47.99	139.02	2.90	0.1	2.98	4.6	645.0
3.01	45.0	48.00	139.50	2.91	0.1	2.98	4.7	648.6
3.02	45.0	48.01	139.98	2.92	0.1	2.98	4.7	652.3
3.03	45.0	48.02	140.46	2.93	0.1	2.98	4.7	655.9
3.04	45.0	48.03	140.94	2.93	0.1	2.98	4.7	658.1
3.05	45.0	48.04	141.42	2.94	0.1	2.98	4.7	661.8
3.06	45.0	48.05	141.90	2.95	0.1	2.98	4.7	665.5
3.07	45.0	48.06	142.38	2.96	0.1	2.98	4.7	669.1
3.08	45.0	48.07	142.86	2.97	0.1	2.98	4.7	672.8
3.09	45.0	48.08	143.34	2.98	0.1	2.98	4.7	676.5
3.10	45.0	48.09	143.82	2.99	0.1	2.98	4.7	680.2
3.11	45.0	48.10	144.31	3.00	0.1	2.98	4.7	684.0
3.12	45.0	48.11	144.79	3.01	0.1	2.98	4.8	687.7
3.13	45.0	48.12	145.27	3.02	0.1	2.98	4.8	691.4
3.14	45.0	48.13	145.75	3.03	0.1	2.98	4.8	695.2
3.15	45.0	48.14	146.23	3.04	0.1	2.98	4.8	698.9
3.16	45.0	48.15	146.71	3.05	0.1	2.98	4.8	704.2
3.17	45.0	48.16	147.19	3.06	0.1	2.98	4.8	707.9
3.18	45.0	48.17	147.67	3.07	0.1	2.98	4.8	711.7
3.19	45.0	48.18	148.16	3.08	0.1	2.98	4.8	715.6
3.20	45.0	48.19	148.64	3.08	0.1	2.98	4.8	717.9
3.21	45.0	48.20	149.12	3.09	0.1	2.98	4.8	721.7

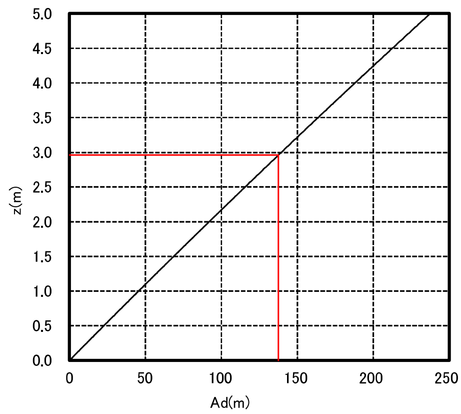
2.96	45.0	47.96	137.58	2.87	0.1	2.98	4.6	632.8



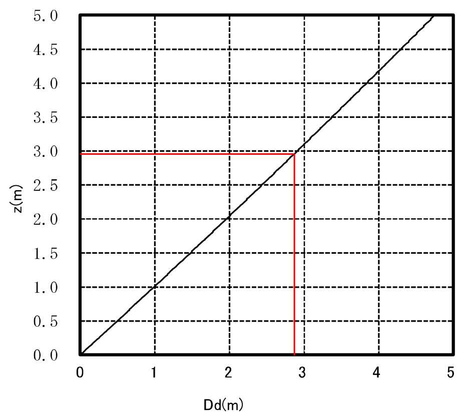
Q_{sp}-Z Curve



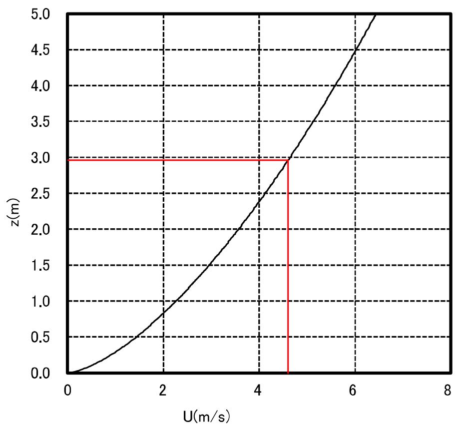
B_{da}-z Curve



A_d-Z Curve



Dd–Z Curve



U–Z Curve

4. Design water depth※1

The design water depth is the largest of the values from the calculation results of below chart.
Therefore, since the value of overflow depth in regards to the discharge bulked with sediment has the largest value, the design water depth is determined to be 3.3m.

Table 1 Overflow water depth calculated by each method

Item	Depth(m)
Value of the overflow depth of debris flow peak discharge	3.0
Value of overflow depth of discharge considering sediment content	3.3
Maximum boulder size	2.5

The freeboard is set based on Table 2. However, the freeboard height shall be designed according to the riverbed slope and freeboard height ratio to the design water depth must not be less than the value presented in Table 3. The riverbed slope referred here is the design sediment deposition slope.

Table 2 Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6 m
200 ～ 500 m^3/s	0.8 m
500 m^3/s or more	1.0 m

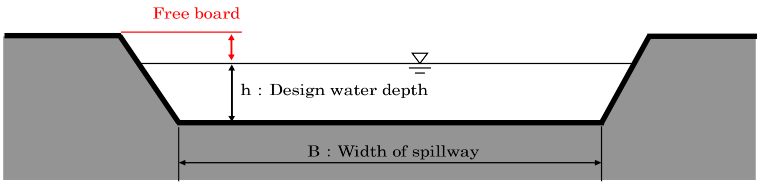


Table 3 Minimum value of the ratio of the freeboard height to the design water depth by riverbed slope

Riverbed slope	(Freeboard height)/(Design water depth)
1/10 or more	0.50
1/10～1/30	0.40
1/30～1/50	0.30
1/50～1/70	0.25

Design deposit surface slope : 1/19.2
Lowest value of (Freeboard height)/(Design water depth) : 0.4

Since the design discharge is 472.5m3/s, the freeboard is 0.8 m
(Freeboard height)/(Design water depth) is 0.8m / 3.3m = 0.24 ＜ 0.4
(Minimum required freeboard height = 1.32m)

Therefore, the freeboard height adopts 1.4m

As a result of the above, Spillway height is 3.3 + 1.4 = 4.7 m

※1 Source : SNI 2851-2021_Desain Sabodam

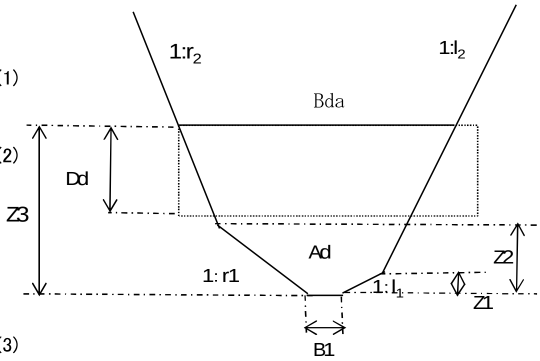
5. Specifications of debris flow

5.1 Setting of debris flow velocity and depth based on debris flow peak discharge

5.1.1 Debris flow cross-section

n_{r1}	:	Flow section right bank slope (bottom)	1.00
n_{r2}	:	Flow section right bank slope (top)	1.00
n_{l1}	:	Flow section left bank slope (bottom)	1.00
n_{l2}	:	Flow section left bank slope (top)	1.00
z	:	Debris flow surface water level	
Bda	:	Width of the flow	

Bda= $n_{r1} \times z + n_{l1} \times z$ (0<z<z1)
$n_{r1} \times z + n_{l2} \times z$ (z1<z<z2)
$n_{r2} \times z + n_{l2} \times z$ (z2<z) ・・・(1)



5.1.2 debris flow depth (Dd)

Dd= Ad ／ Bda ・・・(2)

Ad : Cross-sectional area of debris flow peal

Ad= Ad1 (0<z<z1)
Ad2 (z1<z<z2)
Ad3 (z2<z)

5.1.3 Debris flow velocity (U)

$$U = \frac{1}{Kn} \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2} \quad \cdots(3)$$

Kn	:	Roughness coefficient	0.10		
θ	:	Riverbed surface slope	3.58	(degrees)	(i=1／16.0)

5.1.4 Calculate debris flow velocity and depth

$Q_{special}$= U・Ad ・・・(4)

From formulas (1), (2), (3), and (4), when z is calculated when Qspecial matches Qsp, z is as follows.

z= 1.74m

Calculating debris flow velocity (U) and depth (Dd) from the result of z above, U and Dd are as follows.

Dd= 1.71m , U= 3.57m/s

5.2 Unit weight of debris flow (γd)

γd= (σ×Cd)+{ρ×(1－Cd)}
= {(2,600 × 0.30)+(1,200 × (1－ 0.30)}×9.81/1000
= 15.89 (kN/m^3)

5.3 Drag force of debris flow (F)

$$F = \alpha \cdot \frac{\gamma_d}{g} \cdot Dd \cdot U^2$$

$$= 1.0 \times \frac{15.89}{9.81} \times 1.71 \times 3.57^2$$

= 35.31 (KN／m)

5.4 Unit weight of debris flow in water (ρdi)

ρdi=γd－Wo
= 15.89 － 11.77 = 4.12 (kN/m3)

5.5 Unit weight of sediment in water (γs)

γs=C*(σ－ρ)×g/1000
= 0.6 ×(2,600 － 1,200)×9.81/1000 = 8.24 (kN/m3)

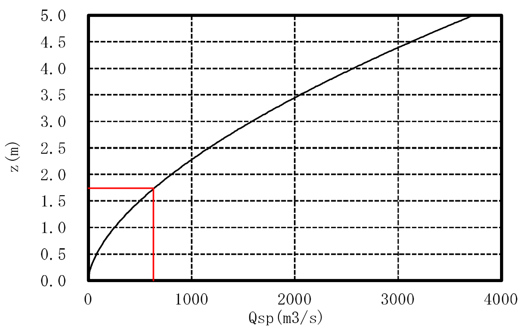
5. 6 Debris flow specifications over Riverbed

Debris flow discharge (Qsp) 630.0 m^3/s
Riverbed surface slope 1/16.0

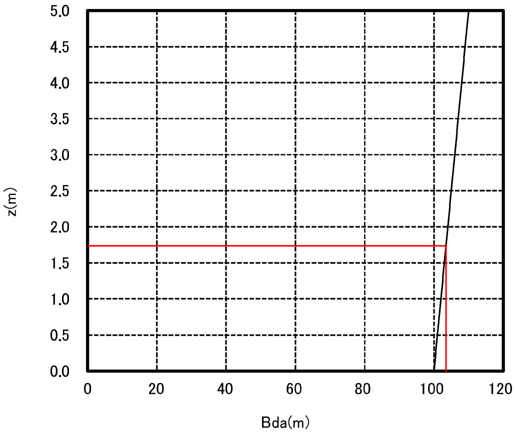
$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
1.49	100.0	103.00	151.22	1.47	0.1	3.58	3.23	488.4
1.50	100.0	103.00	152.25	1.48	0.1	3.58	3.25	494.8
1.51	100.0	103.00	153.28	1.49	0.1	3.58	3.26	499.6
1.52	100.0	103.00	154.31	1.50	0.1	3.58	3.27	504.5
1.53	100.0	103.10	155.34	1.51	0.1	3.58	3.29	511.0
1.54	100.0	103.10	156.37	1.52	0.1	3.58	3.30	516.0
1.55	100.0	103.10	157.40	1.53	0.1	3.58	3.32	522.5
1.56	100.0	103.10	158.43	1.54	0.1	3.58	3.33	527.5
1.57	100.0	103.10	159.46	1.55	0.1	3.58	3.35	534.1
1.58	100.0	103.20	160.50	1.56	0.1	3.58	3.36	539.2
1.59	100.0	103.20	161.53	1.57	0.1	3.58	3.38	545.9
1.60	100.0	103.20	162.56	1.58	0.1	3.58	3.39	551.0
1.61	100.0	103.20	163.59	1.58	0.1	3.58	3.39	554.5
1.62	100.0	103.20	164.62	1.59	0.1	3.58	3.40	559.7
1.63	100.0	103.30	165.66	1.60	0.1	3.58	3.42	566.5
1.64	100.0	103.30	166.69	1.61	0.1	3.58	3.43	571.7
1.65	100.0	103.30	167.72	1.62	0.1	3.58	3.45	578.6
1.66	100.0	103.30	168.76	1.63	0.1	3.58	3.46	583.9
1.67	100.0	103.30	169.79	1.64	0.1	3.58	3.48	590.8
1.68	100.0	103.40	170.82	1.65	0.1	3.58	3.49	596.1
1.69	100.0	103.40	171.86	1.66	0.1	3.58	3.50	601.5
1.70	100.0	103.40	172.89	1.67	0.1	3.58	3.52	608.5
1.71	100.0	103.40	173.92	1.68	0.1	3.58	3.53	613.9
1.72	100.0	103.40	174.96	1.69	0.1	3.58	3.55	621.1
1.73	100.0	103.50	175.99	1.70	0.1	3.58	3.56	626.5
1.74	100.0	103.50	177.03	1.71	0.1	3.58	3.57	631.9
1.75	100.0	103.50	178.06	1.72	0.1	3.58	3.59	639.2
1.76	100.0	103.50	179.10	1.73	0.1	3.58	3.60	644.7
1.77	100.0	103.50	180.13	1.74	0.1	3.58	3.61	650.2
1.78	100.0	103.60	181.17	1.75	0.1	3.58	3.63	657.6
1.79	100.0	103.60	182.20	1.76	0.1	3.58	3.64	663.2
1.80	100.0	103.60	183.24	1.77	0.1	3.58	3.66	670.6
1.81	100.0	103.60	184.28	1.78	0.1	3.58	3.67	676.3
1.82	100.0	103.60	185.31	1.79	0.1	3.58	3.68	681.9
1.83	100.0	103.70	186.35	1.80	0.1	3.58	3.70	689.4
1.84	100.0	103.70	187.39	1.81	0.1	3.58	3.71	695.2
1.85	100.0	103.70	188.42	1.82	0.1	3.58	3.72	700.9
1.86	100.0	103.70	189.46	1.83	0.1	3.58	3.74	708.5
1.87	100.0	103.70	190.50	1.84	0.1	3.58	3.75	714.3
1.88	100.0	103.80	191.53	1.85	0.1	3.58	3.77	722.0
1.89	100.0	103.80	192.57	1.86	0.1	3.58	3.78	727.9
1.90	100.0	103.80	193.61	1.87	0.1	3.58	3.79	733.7
1.91	100.0	103.80	194.65	1.87	0.1	3.58	3.79	737.7
1.92	100.0	103.80	195.69	1.88	0.1	3.58	3.81	745.5
1.93	100.0	103.90	196.72	1.89	0.1	3.58	3.82	751.4
1.94	100.0	103.90	197.76	1.90	0.1	3.58	3.83	757.4
1.95	100.0	103.90	198.80	1.91	0.1	3.58	3.85	765.3
1.96	100.0	103.90	199.84	1.92	0.1	3.58	3.86	771.3
1.97	100.0	103.90	200.88	1.93	0.1	3.58	3.87	777.4
1.98	100.0	104.00	201.92	1.94	0.1	3.58	3.89	785.4
1.99	100.0	104.00	202.96	1.95	0.1	3.58	3.90	791.5

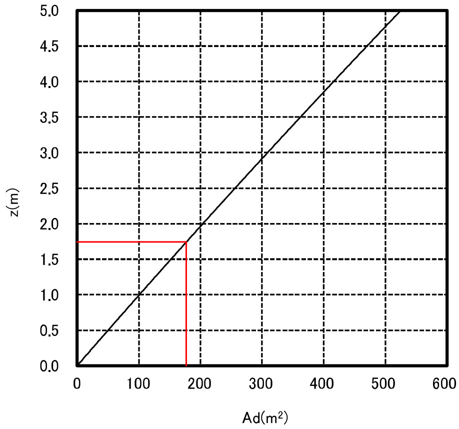
1.74	100.0	103.48	177.03	1.71	0.1	3.58	3.57	631.9



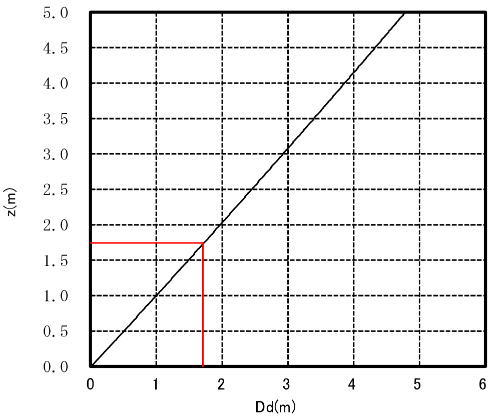
Q_{sp}-Z Curve



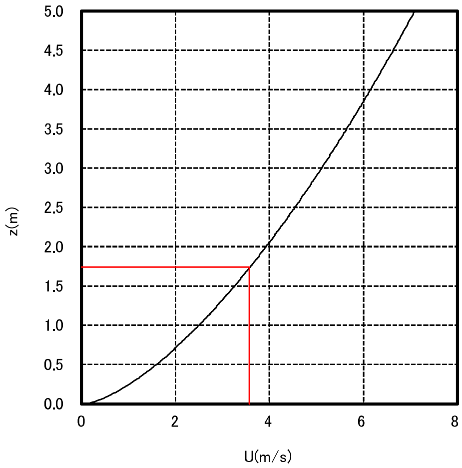
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

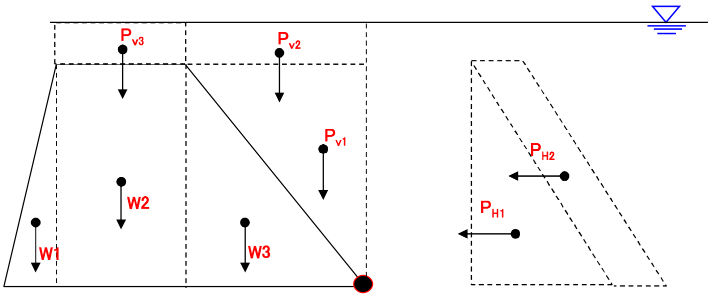
6. Stability analysis

6.1 Result of Stability analysis (overflow section)

During flooding					
Stability against overturning	e	1.723	≦	1.958	OK
Stability against sliding	n	7.26	≧	4.0	OK
Maximum ground reaction force	σ max	422.7	≦	1200	OK
Minimum ground reaction force	σ min	27.1	≧	0	OK
During debris flow					
Stability against overturning	e	0.999	≦	1.958	OK
Stability against sliding	n	9.34	≧	4.0	OK
Maximum ground reaction force	σ max	310.5	≦	1200	OK
Minimum ground reaction force	σ min	100.8	≧	0	OK

6. 2 Verification of stability of main body (overflow section)

6.2.1 During flooding



Schematic diagram of design external forces

Remarks : Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 2.30 × 11.50 × 20.00 11.75 − 2/3 × 2.30	264.500		10.217	2,702.308
W2	6.00 × 11.50 × 20.00 3.45 + 1/2 × 6.00	1,380.000		6.450	8,901.000
W3	1/2 × 3.45 × 11.50 × 20.00 2/3 × 3.45	396.750		2.300	912.525
Hydrostatic pressure P_{v1}	1/2 × 3.45 × 11.50 × 11.77 1/3 × 3.45	233.487		1.150	268.510
P_{v2}	3.45 × 3.30 × 11.77 1/2 × 3.45	134.001		1.725	231.153
P_{v3}	6.00 × 3.30 × 11.77 3.45 + 1/2 × 6.00	233.046		6.450	1,503.147
P_{H1}	1/2 × 11.50 × 11.50 × 11.77 1/3 × 11.50		778.291	3.833	2,983.450
P_{H2}	3.30 × 11.50 × 11.77 1/2 × 11.50		446.672	5.750	2,568.361
TOTAL		2,641.785	1,224.963		20,070.454

Stability against overturning

Resultant force position
$$d=\frac{\Sigma M}{\Sigma V}=\frac{20070.454}{2641.785}=7.597 \quad m$$

Eccentric distance
$$e= d-B/2 = 7.597 - 11.750 / 2 = 1.723 \quad m$$
$$e= 1.723 \ m \leqq B/6= 1.958 \ m$$

OK

Stability against sliding

$$n=\frac{f\times \Sigma V+\tau \times L}{H}$$
$$=\frac{0.700 \times 2641.785 + 600.0 \times 11.750}{1224.963} = 7.26$$
$$n= 7.26 \geqq 4.0$$

OK

Stability of bearing capacity

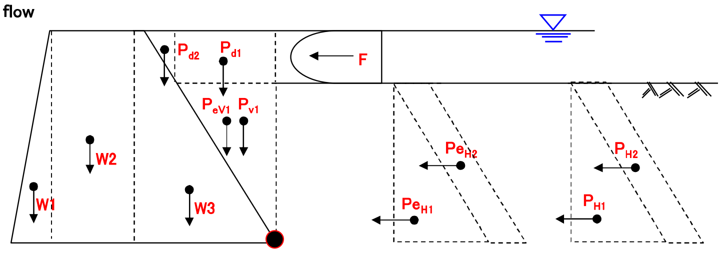
Minimum ground reaction force
$$\sigma min=\frac{\Sigma V}{B}\times\left(1-\frac{6\times e}{B}\right)$$
$$=\frac{2641.79}{11.75}\times\left(1-\frac{6\times 1.72}{11.75}\right)=27.1 \ kN/m^2 \geqq 0 \ kN/m^2$$

OK

Maximum ground reaction force
$$\sigma max=\frac{\Sigma V}{B}\times\left(1+\frac{6\times e}{B}\right)$$
$$=\frac{2641.79}{11.75}\times\left(1+\frac{6\times 1.72}{11.75}\right)=422.7 \ kN/m^2 \leqq 1200 \ kN/m^2$$

OK

6.2.2 During bebris flow



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN·m)
Self-weight of dam W1	1/2 × 2.30 × 11.50 × 20.00 11.75 − 2/3 × 2.30	264.500		10.217	2,702.308
W2	6.00 × 11.50 × 20.00 3.45 + 1/2 × 6.00	1,380.000		6.450	8,901.000
W3	1/2 × 3.45 × 11.50 × 20.00 2/3 × 3.45	396.750		2.300	912.525
Hydrostatic pressure P_{V1}	1/2 × 2.94 × 9.79 × 11.77 1/3 × 2.94	169.213		0.979	165.659
P_{H1}	1/2 × 9.79 × 9.79 × 11.77 1/3 × 9.79		564.043	3.263	1,840.659
P_{H2}	1.71 × 9.79 × 11.77 1/2 × 9.79		197.040	4.895	964.513
Earth pressure P_{ev1}	1/2 × 2.94 × 9.79 × 8.24 1/3 × 2.94	118.463		0.979	115.976
P_{eH1}	1/2 × 0.30 × 9.79 ^{2}× 8.24 1/3 × 9.79		118.463	3.263	386.585
P_{eH2}	0.30 × 1.71 × 9.79 × 4.12 1/2 × 9.79		20.690	4.895	101.278
Mass of debris flow Pd1	2.94 × 1.71 × 15.89 1/2 × 2.94	79.804		1.469	117.192
Pd2	1/2 × 0.51 × 1.71 × 15.89 3.45 − 2/3 × 0.51	6.970		3.108	21.663
Drag force of debris flow F	1.00 × 15.89 × 3.57 ^{2} × 1.71 /9.81 9.79 + 1/2 × 1.71		35.301	10.645	375.780
TOTAL		2,415.700	935.537		16,605.138

Stability against overturning

Resultant force position $d=\dfrac{\Sigma M}{\Sigma V}=\dfrac{16605.138}{2415.700}=6.874$ m

Eccentric distance

$e=d-B/2 = 6.874 - 11.750 / 2 = 0.999$ m

$e= 0.999$ m $\leqq B/6= 1.958$ m .... OK

Stability against sliding

$$n=\frac{f\times \Sigma V+\tau \times L}{H}$$

$$=\frac{0.700 \times 2415.700 + 600.0 \times 11.75}{935.537} = 9.34$$

$n= 9.34 \geqq 4.0$ .... OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min=\frac{\Sigma V}{B}\times\left(1-\frac{6\times e}{B}\right)$$

$$=\frac{2415.70}{11.75}\times\left(1-\frac{6\times 1.00}{11.75}\right)=100.8\ \mathrm{kN/m^2} \geqq 0\ \mathrm{kN/m^2}$$

OK

Maximum ground reaction force

$$\sigma max=\frac{\Sigma V}{B}\times\left(1+\frac{6\times e}{B}\right)$$

$$=\frac{2415.70}{11.75}\times\left(1+\frac{6\times 1.00}{11.75}\right)=310.5\ \mathrm{kN/m^2} \leqq 1200\ \mathrm{kN/m^2}$$

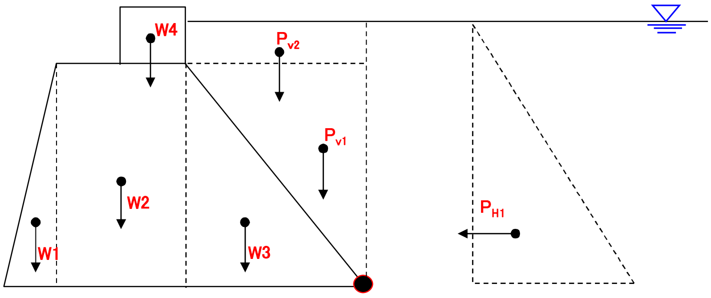
.... OK

6.3 Result of Stability analysis (non-overflow section)

During flooding					
Stability against overturning	e	1.008	≦	2.433	OK
Stability against sliding	n	8.77	≧	4.0	OK
Maximum ground reaction force	σ max	352.4	≦	1200	OK
Minimum ground reaction force	σ min	146.0	≧	0	OK
During debris flow					
Stability against overturning	e	0.612	≦	2.433	OK
Stability against sliding	n	9.21	≧	4.0	OK
Maximum ground reaction force	σ max	318.9	≦	1200	OK
Minimum ground reaction force	σ min	190.8	≧	0	OK

6.4 Verification of stability of main body (non-overflow section)

6.4.1 During flooding



Schematic diagram of design external forces

Remarks : Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 2.30 × 11.50 × 20.00 14.60 − 2/3 × 2.30	264.500		13.067	3,456.133
W2	10.00 × 11.50 × 20.00 2.30 + 1/2 × 10.00	2,300.000		7.300	16,790.000
W3	1/2 × 2.30 × 11.50 × 20.00 2/3 × 2.30	264.500		1.533	405.567
W4	4.70 × 6.00 × 20.00 2.30 + 6.00 × 1/2	564.000		5.300	2,989.200
Hydrostatic pressure P_{V1}	1/2 × 2.30 × 11.50 × 11.77 1/3 × 2.30	155.658		0.767	119.338
P_{V2}	2.30 × 3.30 × 11.77 1/2 × 2.30	89.334		1.150	102.734
P_{H1}	1/2 × 14.80 × 14.80 × 11.77 1/3 × 14.80		1,289.050	4.933	6,359.315
TOTAL		3,637.993	1,289.050		30,222.287

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{30222.287}{3637.993} = 8.307$ m

Eccentric distance
$e = d - B/2 = 8.307 - 14.600 / 2 = 1.008$ m
$e = 1.008$ m $\leqq B/6 = 2.433$ m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.700 \times 3637.993 + 600.0 \times 14.600}{1289.050} = 8.77$$

$n = 8.77 \geqq 4.0$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma\, min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{3637.99}{14.60} \times \left(1 - \frac{6 \times 1.01}{14.60}\right) = 146.0 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

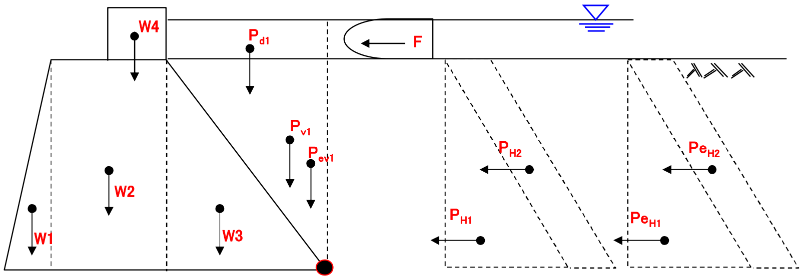
Maximum ground reaction force

$$\sigma\, max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{3637.99}{14.60} \times \left(1 + \frac{6 \times 1.01}{14.60}\right) = 352.4 \text{ kN/m}^2 \leqq 1200 \text{ kN/m}^2$$

OK

6.4.2 During bebris flow



Schematic diagram of design external forces

Remarks：Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN•m)
Self-weight of dam W1	1/2 × 2.30 × 11.50 × 20.00 14.60 − 2/3 × 2.30	264.500		13.067	3,456.133
W2	10.00 × 11.50 × 20.00 2.30 ＋ 1/2 × 10.00	2,300.000		7.300	16,790.000
W3	1/2 × 2.30 × 11.50 × 20.00 2/3 × 2.30	264.500		1.533	405.567
W4	4.70 × 6.00 × 20.00 2.30 ＋ 6.00 × 1/2	564.000		5.300	2,989.200
Hydrostatic pressure P_{V1}	1/2 × 2.30 × 11.50 × 11.77 1/3 × 2.30	155.658		0.767	119.338
P_{H1}	1/2 × 11.50 × 11.50 × 11.77 1/3 × 11.50		778.291	3.833	2,983.450
P_{H2}	1.71 × 11.50 × 11.77 1/2 × 11.50		231.457	5.750	1,330.878
Earth pressure P_{ev1}	1/2 × 2.30 × 11.50 × 8.24 1/3 × 2.30	108.974		0.767	83.547
P_{eH1}	1/2 × 0.30 × 11.50 ^{2}× 8.24 1/3 × 11.50		163.461	3.833	626.601
P_{eH2}	0.30 × 1.71 × 11.50 × 4.12 1/2 × 11.50		24.310	5.750	139.783
Mass of debris flow Pd1	2.30 × 1.71 × 15.89 1/2 × 2.30	62.495		1.150	71.869
Drag force of debris flow F	1.00 × 15.89 × 3.57 ^{2} × 1.71 /9.81 11.50 ＋ 1/2 × 1.71		35.301	12.355	436.144
TOTAL		3,720.127	1,232.820		29,432.510

Stability against overturning

Resultant force position $d=\dfrac{\Sigma M}{\Sigma V}=\dfrac{29432.510}{3720.127}=7.912$ m

Eccentric distance $e= d-B/2 = 7.912 - 14.600 / 2 = 0.612$ m

$e= 0.612$ m $\leqq B/6= 2.433$ m ..... OK

Stability against sliding

$$n=\frac{f\times \Sigma V+\tau\times L}{H}$$

$$=\frac{0.700\times 3720.127+600.0\times 14.600}{1232.820}=9.21$$

$n= 9.21 \geqq 4.0$

OK

Stability of bearing capacity

Minimum ground reaction force $\sigma min=\dfrac{\Sigma V}{B}\times\left(1-\dfrac{6\times e}{B}\right)$

$$=\frac{3720.13}{14.60}\times\left(1-\frac{6\times 0.61}{14.60}\right)=190.8 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

Maximum ground reaction force $\sigma max=\dfrac{\Sigma V}{B}\times\left(1+\dfrac{6\times e}{B}\right)$

$$=\frac{3720.13}{14.60}\times\left(1+\frac{6\times 0.61}{14.60}\right)=318.9 \text{ kN/m}^2 \leqq 1200 \text{ kN/m}^2$$

OK

7. Downstream riverbed protection works※1

7.1 Thickness of the apron

Thickness of the apron is calculated as follow.

Empirical formula (If there is the water cushion)

$$t = 0.1 \times (0.6H_1 + 3h_3 - 1.0)$$

Where,
- t : Thickness of the apron (m)
- H_1 : Height from the top of the apron to the base of the spillway (m)
- h_3 : Design water depth(m)

If H : Dam height(m)

Then $H_1 = H - t$

$$t = 0.1 \times (0.6(H-t) + 3h_3 - 1.0)$$

$$= -0.06t + (0.1 \times (0.6H + 3h_3 - 1.0)$$

Therefore,

$$t = 0.1 \times (0.6H + 3h_3 - 1.0) / 1.06$$

In case of the planned main dam,

Then H = 11.5 m

h_3 = 3.3 m

$$t = 0.1 \times (0.6 \times 11.5 + 3 \times 3.3 - 1.0) / 1.06$$

$$= 1.49057 \text{ m}$$

t = 1.5 m

⇒ Adopted value : 2.0 m

7.2 Overlap height

Overlap height between main-dam and sub-dam is calculated as follow.

Empirical formula

$H_2 = (1/3 \sim 1/4)\ H$

Where,
- H : Dam height(m)
- H_2 : Overlap height(m)

In addition, there are many cases where 1/3 to 1/4 is used as follow.

If the foundation is rock : 1/4

If the foundation is gravel : 1/3

H : Dam height (m)	Foundation	Coefficient	H_2 : Overlap height	Adoption
11.5	Gravel	1/3	3.83	3.9

⇒ Adopted value : 4.0 m

7.3 Length of the apron

Length of the apron (L) is calculated as follow.

$$L = (1.5 \sim 2.0) \times (H_1 + H_3)$$

$L = 1.5 \times (H1 + H3) =$ 19.2 m

$L = 2.0 \times (H1 + H3) =$ 25.6 m

⇒ Adopted value : 25.0 m

※1 Source : SNI 2851-2021_Desain Sabodam

8. Ability to flow down of the opening section

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Number of opening section		Parts	7
Peak discharge of water	Q	m^3/s	315
Target discharge amount per part		$m^3/s/part$	45.00
Coefficient	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Width of opening section	B1	m	3.0

The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Q = \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1 + 2B_2) \times Dh^{3/2}$$

Where, Dh : Water depth (m)
B_2 : Width of overflow (m)

In case of Dh=4.1

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 3.0 + 2 \times 3.0) \times 4.1\hat{}(3/2) = 44.12 \quad < 45.00$$

・・・・・NG

In case of Dh=4.2

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 3.0 + 2 \times 3.0) \times 4.2\hat{}(3/2) = 45.75 \quad \geqq 45.00$$

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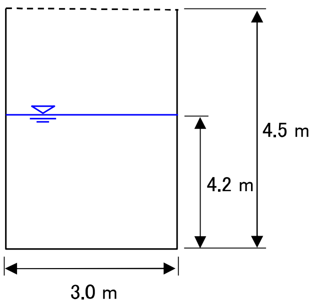
Therefore,
Dh ＝ 4.2 m



Schematic diagram of opening section

The overflow water depth is 4.2m, and the opening height is 4.5m, so the target water flow can flow down only through the opening section.

KD Leprak 3

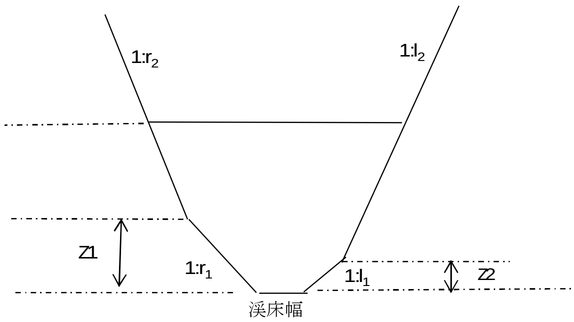
Stability calculation of S4-3 KD Leprak3
(Dam height H<15m)

1. Design specifications

1.1 Design condition list

①Topography, geology, facility shape conditions

Symbol	Item	Unit	Value
	Topographic condition		
	Width of Riverbed	m	100.0
$1:r_1$	Flow section right bank slope (bottom)		1.0
$1:r_2$	Flow section right bank slope (top)		1.0
$1:l_1$	Flow section left bank slope (bottom)		1.0
$1:l_2$	Flow section left bank slope (top)		1.0
θ	Riverbed surface slope		1/33.3
	Flow rate conditions		
A	Target basin area	km^2	25.80
P_{24}	24-hour(day) rainfall	mm	251.0
	Facility shape conditions		
H	Dam height	m	7.0
B	Crest width	m	3.0
b1	Road width	m	0.0
b2	Wing crest width	m	3.0
m	Upstream slope(overflow section)		0.50
m	Upstream slope(non-overflow section)		0.50
n	Downstream slope		0.20
B1	Base width of spillway	m	130.0
	Wing edge		0.5
	Sediment conditions		
d_{95}	Maximum boulder size	m	2.5
	Ground conditions		
τ_0	Shear strength		0
f	Friction coefficient		0.60
qu	Allowable bearing capacity	kN/m^2	600
n	Safety factor against sliding		1.2



②Design constant

We	Apparent unit weight of concrete	kN/m^3	20
We	Unit weight of concrete	kN/m^3	22.6
Wo	Unit weight of water	kN/m^3	11.77
σ	Gravel density	kg/m^3	2,600
ρ	Water density	kg/m^3	1,200
K_{p1}	Coefficient		120
C	Coefficient of overflow		0.6
g	Gravitational acceleration	m/s^2	9.81
C_*	Volumetric concentration of sediment		0.6
Kn	Roughness coefficient(front part)		0.1
C	Coefficient of earth pressure		0.3
ϕ	Internal friction angle of sediment	degree	35.0
τ c	Shear strength of concrete		2,760

Regarding the shear strength, the smaller value in between the dambody and the ground is used in calculation.

1.2 Apparent unit weight of concrete ※1

Symbol	Item	Unit	Value
B1	Base width of spillway	m	130.0
b	Width of opening section	m	3.0
h	Height of opening section	m	4.0
h1	Slab above the opening	m	0.0
	Number of openings	parts	14
W	Dam body volume except opening	m3	1,995
Vc	Dam body volume including opening	m3	2,230
Wc	Unit weight of concrete	kN/m3	22.6
Wγc	Apparent unit weight of concrete	kN/m3	20.22

The apparent unit weight of concrete adopts 20.0 kN/m3

⇒ Adopted value : 20.0 kN/m3

3) If the permeable part is made of concrete, the self-weight of the dam body is calculated using the dam body block volume (V_c) which is calculated by assuming the overflow section as a closed structure, and the dam body block weight (W_{rc}) which is calculated by assuming the overflow section as an open structure (Figure 11). Further, it is noted that the dam body block refers to the concrete block in the permeable part, not the concrete block used at the time of casting.

$$\gamma_{rc} = W_{rc} / V_C \quad \text{......(6)}$$

Where,

γ_{rc} : apparent unit weight of concrete (kN/m^3),

W_{rc} : dam body block weight (kN) calculated by assuming the overflow section as an open structure with concrete,

V_c : dam body block volume (m^3) calculated by assuming the overflow section as a closed structure.

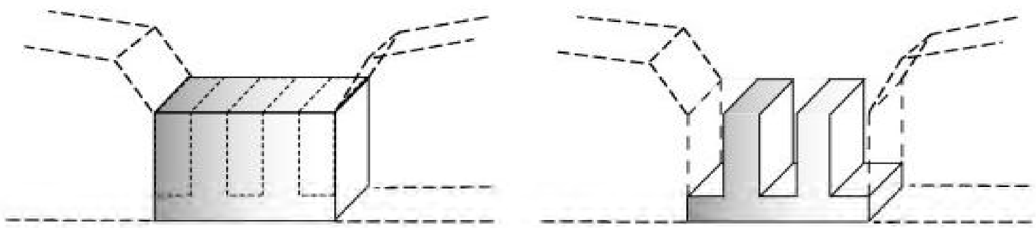


Figure 11 Dam body volume of spillway of the slit part

4) Combinations of design external forces excluding the self-weight of dam body are as shown in Table 4.

※1 Source : Technical Guideline for Designing Sabo Facilities

2. Calculation of flow discharge considering sediment content (1.5Qp)

2. 1 Flow discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Target basin area	A	km^2	25.80

The flow discharge considering sediment content is calculated by under rainfall(251mm/24h) with the design exceedance probability.

The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Q_p = \frac{1}{3.6} \times K_{f1} \times Pa \times A$$ ※1

Qp : Peak discharge of water without sediment(m^3/s)

Pa : Average rainfall intensity during the flood concentration time(mm/hr)

A : Target basin area(km^2)

K_{f1} : Coefficient

Therefore, the peak discharge of water without sediment is as follow.

$$Q_p = 371 \quad (m^3/s)$$

Flow discharge considering sediment content is 1.5 times the peak discharge of water without sediment, and calculated as follow.

$$Q = 1.5 \times Q_p$$ ※1

$$= 1.5 \times 371$$

$$= 556.5 \quad (m^3/s)$$

※1 Source : Technical Guideline for Establishing Sabo Master Plan

2.2 Calculation of overflow depth for discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Discharge of water considering sediment content	Q	m^3/s	556.50
Coefficient of discharge	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Base width of the spillway	B1	m	130.0
Wing edge slope	m		0.5

The overflow depth for discharge considering sediment content is calculated as follow.

$$Q = \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1 + 2B_2) \times Dh^{3/2}$$ ※1

Where, Dh : Overflow depth (m)

B_2 : Width of overflow (m)

In case of Dh=1.7

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 130.0 + 2 \times 131.7) \times 1.7\hat{}(3/2) = 513.20 \quad < 556.50$$

・・・・・NG

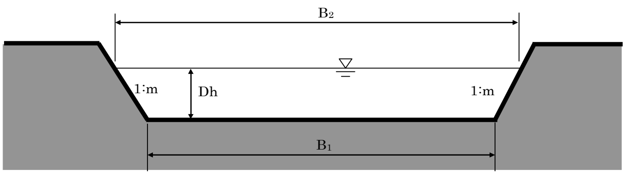
In case of Dh=1.8

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 130.0 + 2 \times 131.8) \times 1.8\hat{}(3/2) = 559.31 \quad \geqq 556.50$$

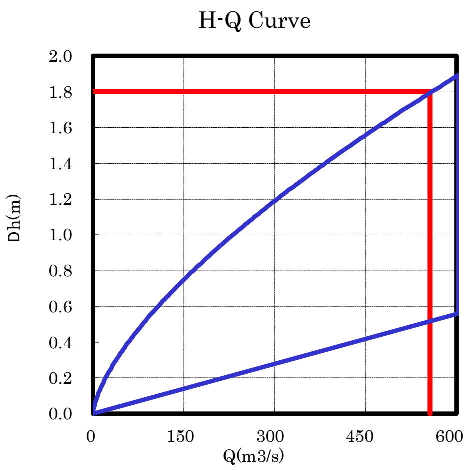
・・・・・OK

Therefore,

Dh ＝ 1.8 m



Schematic diagram of the cross-sectional shape of Spillway



※1 Source : SNI 2851-2021_Desain Saboda

3. Calculation of debris flow peak discharge (Qsp)

3.1 Debris flow peak discharge (Qsp) ※1

Specifications list

Symbol	Item	Unit	Value
σ	Density of gravel	kg/m^3	2,600
ρ	Density of water	kg/m^3	1,200
Wo	Unit weight of the water	kN/m^3	11.77
ϕ	Internal friction angle of the deposited soil	degrees	35.0
d	Maximum boulder size	m	2.50
$\tan\theta$	Riverbed surface slope		1/33.3
C_*	Volumetric concentration of deposited sediment		0.60
Kn	Roughness coefficient (the front part of debris flow)		0.10
B	Width of riverbed	m	100.0

3.1.1 Concentration of debris flow (Takahashi's Equation)

$$Cd = \frac{\rho \cdot \tan\theta}{(\sigma-\rho)\cdot(\tan\phi - \tan\theta)}$$

Where,

ρ	;	Density of water	1,200	(Kg/m^3)
θ	;	Riverbed surface slope	1.72	(degrees) (i=1/33.3)
σ	;	Density of gravel	2,600	(Kg/m^3)
ϕ	;	Internal friction angle of sediment deposited on riverbed	35.0	(degrees)

$$= \frac{1{,}200 \times \tan 1.72}{(2{,}600 - 1{,}200) \times (\tan 35.0 - \tan 1.72)}$$

$$= 0.04$$

When the calculated value (Cd) is larger than 0.9C*, , Cd is assumed to be equal to 0.9C*. Meanwhile, if the calculated value (Cd) is smaller than 0.3, Cd value is assumed to be 0.3

∴ Cd = 0.30

3.1.2 Debris flow peak discharge (Qsp)

The peak discharge of debris flow Qsp (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Qp (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp$$

Where,

Cd	:	Concentration of debris flow	=	0.30	
C*	:	Volumetric concentration of sediment deposited on riverbed	=	0.60	
Qp	:	Peak discharge of water	=	371.00	(m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp = \frac{0.60}{0.60 - 0.30} \times 371 = 742.0$$

Qsp= 742.0 (m^3/sec)

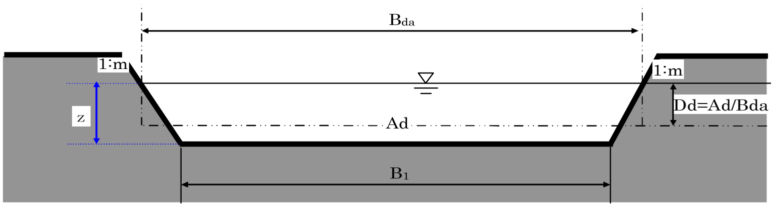
※1 Source : SNI 2851-2021_Desain Sabodam

3.2 Value of the overflow depth for debris flow peak discharge (Qsp).

3.2.1 Calculation of overflow depth ※1

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Debris flow discharge	Qsp	m^3/s	742.0
Roughness coefficient	Kn		0.1
Design deposit surface slope	θ	degrees	1.43
Base width of spillway	B1	m	130.0
Wing edge slope	m		0.5



The amount of the debris flow that can flow down through this cross section is determined as follow.

Qsp=U・Ad

Where,

z :	Debris flow surface water level	(m)
U :	Debris flow velocity	(m/s)
Bda :	Width of the flow	(m)
Ad :	Cross-sectional aera of debris flow peak discharge	(m^2)
Dd :	Debris flow depth	(m)

U, Bda, Ad and Dd can be calculated as follow.

U= 1/Kn・$Dd^{2/3}$・$(\sin\theta)^{1/2}$

Dd= Ad/Bda

Ad= (B1+m×z)×z

Bda= B1+2×m×z

If the debris flow depth is 2.16m

Qsp= 2.62 ×283.13

= 741.8 < 742.0 ·····NG

U= 1/0.1 ×2.14 $^{2/3}$× 0.02 $^{1/2}$= 2.62

Dd= 283.13/ 132.16 = 2.14

Ad= (130.0 +0.5× 2.16) ×2.16 = 283.13

Bda= 130.0 +2×0.50 ×2.16 = 132.16

(U,Dd,Ad,Bda：少数第2位四捨五入)

If the debris flow depth is 2.17m

Qsp= 2.63 ×284.45

= 748.1 ≧ 742.0 ·····OK

U= 1/0.1 ×2.15 $^{2/3}$× 0.02 $^{1/2}$= 2.63

Dd= 284.45/ 132.17 = 2.15

Ad= (130.0 +0.5× 2.17) ×2.17 = 284.45

Bda= 130.0 +2×0.50 ×2.17 = 132.17

Therefore, the debris flow depth is adapted as 2.2m

※1 Source : Technical Guideline for Establishing Sabo Master Plan

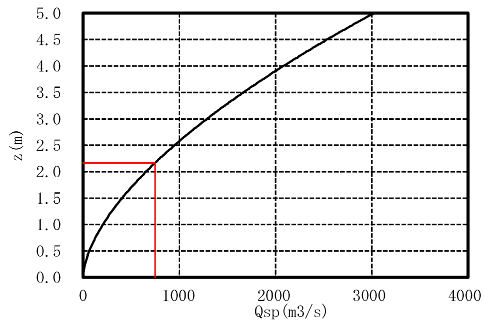
3.2.2 Debris flow specifications over spillway

$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

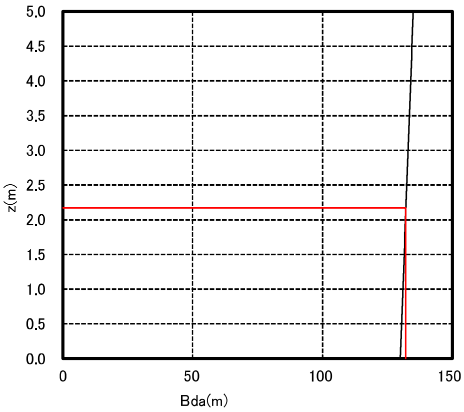
Debris flow discharge (Qsp) 742.0 m^3/s
Design deposit surface slope 1/40.0

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
1.92	130.0	131.92	251.44	1.91	0.1	1.43	2.4	610.9
1.93	130.0	131.93	252.76	1.92	0.1	1.43	2.4	616.7
1.94	130.0	131.94	254.08	1.93	0.1	1.43	2.5	622.4
1.95	130.0	131.95	255.40	1.94	0.1	1.43	2.5	628.2
1.96	130.0	131.96	256.72	1.95	0.1	1.43	2.5	634.0
1.97	130.0	131.97	258.04	1.96	0.1	1.43	2.5	637.3
1.98	130.0	131.98	259.36	1.97	0.1	1.43	2.5	643.2
1.99	130.0	131.99	260.68	1.97	0.1	1.43	2.5	646.4
2.00	130.0	132.00	262.00	1.98	0.1	1.43	2.5	652.3
2.01	130.0	132.01	263.32	1.99	0.1	1.43	2.5	658.3
2.02	130.0	132.02	264.64	2.00	0.1	1.43	2.5	664.2
2.03	130.0	132.03	265.96	2.01	0.1	1.43	2.5	670.2
2.04	130.0	132.04	267.28	2.02	0.1	1.43	2.5	673.5
2.05	130.0	132.05	268.60	2.03	0.1	1.43	2.5	679.5
2.06	130.0	132.06	269.92	2.04	0.1	1.43	2.5	685.5
2.07	130.0	132.07	271.24	2.05	0.1	1.43	2.6	691.6
2.08	130.0	132.08	272.56	2.06	0.1	1.43	2.6	697.7
2.09	130.0	132.09	273.88	2.07	0.1	1.43	2.6	703.8
2.10	130.0	132.10	275.21	2.08	0.1	1.43	2.6	707.2
2.11	130.0	132.11	276.53	2.09	0.1	1.43	2.6	713.4
2.12	130.0	132.12	277.85	2.10	0.1	1.43	2.6	719.6
2.13	130.0	132.13	279.17	2.11	0.1	1.43	2.6	725.8
2.14	130.0	132.14	280.49	2.12	0.1	1.43	2.6	732.0
2.15	130.0	132.15	281.81	2.13	0.1	1.43	2.6	738.3
2.16	130.0	132.16	283.13	2.14	0.1	1.43	2.6	741.8
2.17	130.0	132.17	284.45	2.15	0.1	1.43	2.6	748.1
2.18	130.0	132.17	284.45	2.15	0.1	1.43	2.6	748.1
2.19	130.0	132.18	285.78	2.16	0.1	1.43	2.6	754.4
2.20	130.0	132.19	287.10	2.17	0.1	1.43	2.7	760.8
2.21	130.0	132.20	288.42	2.18	0.1	1.43	2.7	767.1
2.22	130.0	132.21	289.74	2.19	0.1	1.43	2.7	770.7
2.23	130.0	132.22	291.06	2.20	0.1	1.43	2.7	777.1
2.24	130.0	132.23	292.39	2.21	0.1	1.43	2.7	783.6
2.25	130.0	132.24	293.71	2.22	0.1	1.43	2.7	790.0
2.26	130.0	132.25	295.03	2.23	0.1	1.43	2.7	796.5
2.27	130.0	132.26	296.35	2.24	0.1	1.43	2.7	800.1
2.28	130.0	132.27	297.68	2.25	0.1	1.43	2.7	806.7
2.29	130.0	132.28	299.00	2.26	0.1	1.43	2.7	813.2
2.30	130.0	132.29	300.32	2.27	0.1	1.43	2.7	819.8
2.31	130.0	132.30	301.65	2.28	0.1	1.43	2.7	826.5
2.32	130.0	132.31	302.97	2.29	0.1	1.43	2.7	830.1
2.33	130.0	132.32	304.29	2.30	0.1	1.43	2.8	836.7
2.34	130.0	132.33	305.61	2.31	0.1	1.43	2.8	843.4
2.35	130.0	132.34	306.94	2.32	0.1	1.43	2.8	850.2
2.36	130.0	132.35	308.26	2.33	0.1	1.43	2.8	856.9
2.37	130.0	132.36	309.58	2.34	0.1	1.43	2.8	860.6
2.38	130.0	132.37	310.91	2.35	0.1	1.43	2.8	867.4
2.39	130.0	132.38	312.23	2.36	0.1	1.43	2.8	874.2
2.40	130.0	132.39	313.56	2.37	0.1	1.43	2.8	881.1
2.41	130.0	132.40	314.88	2.38	0.1	1.43	2.8	887.9
2.42	130.0	132.41	316.20	2.39	0.1	1.43	2.8	891.6

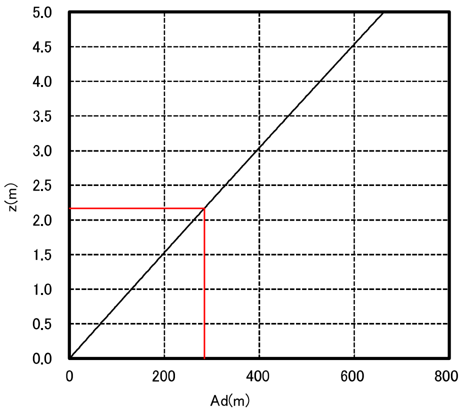
2.17	130.0	132.17	284.45	2.15	0.1	1.43	2.7	748.1



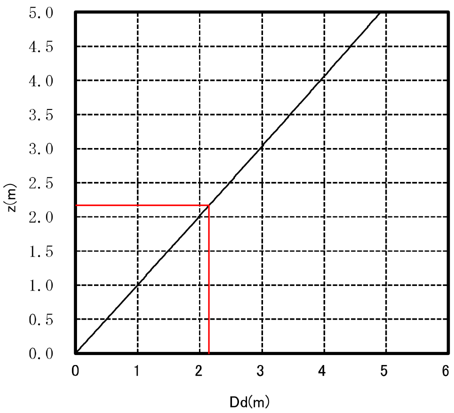
Q_{sp}-Z Curve



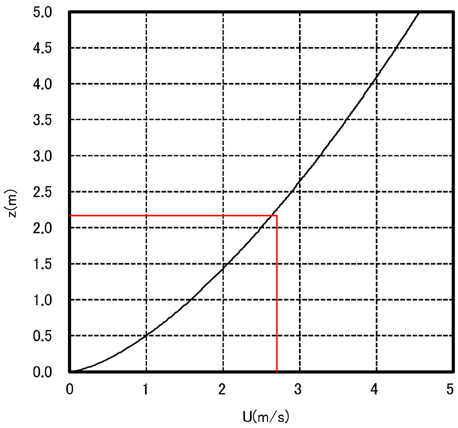
B_{da}-z Curve



A_d-Z Curve



Dd–Z Curve



U–Z Curve

4. Design water depth※1

The design water depth is the largest of the values from the calculation results of Table 1.
Therefore, since the Maximum boulder size has the largest value, the design water depth is determined to be 2.5m.

Table 1 Overflow water depth calculated by each method

Item	Depth(m)
Value of the overflow depth of debris flow peak discharge	2.2
Value of overflow depth of discharge considering sediment content	1.8
Maximum boulder size	2.5

The freeboard is set based on Table 2. However, the freeboard height shall be designed according to the riverbed slope and freeboard height ratio to the design water depth must not be less than the value presented in Table 3. The riverbed slope referred here is the design sediment deposition slope.

Table 2 Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6 m
200 ～ 500 m^3/s	0.8 m
500 m^3/s or more	1.0 m

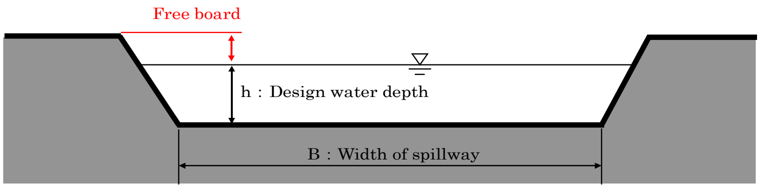


Table 3 Minimum value of the ratio of the freeboard height to the design water depth by riverbed slope

Riverbed slope	(Freeboard height)/(Design water depth)
1/10 or more	0.50
1/10～1/30	0.40
1/30～1/50	0.30
1/50～1/70	0.25

Design deposit surface slope : 1/40
Lowest value of (Freeboard height)/(Design water depth) : 0.3

Since the design discharge is 556.5m3/s, the freeboard is 1.0 m
(Freeboard height)/(Design water depth) is 1m / 2.5m = 0.40 ＞ 0.3
(Minimum required freeboard height = 0.75m)

Therefore, the freeboard height adopts 1.0m

As a result of the above, Spillway height is 2.5 + 1 = 3.5 m

However, the spillway height will be adjusted to match the height of the existing Dike, and a height of 4.5 m will be adopted.

※1 Source : SNI 2851-2021_Desain Sabodam

5. Specifications of debris flow

5.1 Setting of debris flow velocity and depth based on debris flow peak discharge

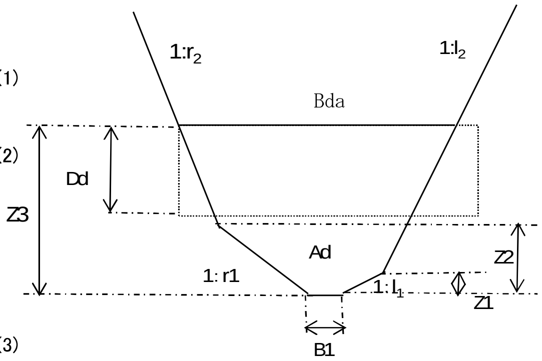
5.1.1 Debris flow cross-section

n_{r1}	:	Flow section right bank slope (bottom)	1.00
n_{r2}	:	Flow section right bank slope (top)	1.00
n_{l1}	:	Flow section left bank slope (bottom)	1.00
n_{l2}	:	Flow section left bank slope (top)	1.00
z	:	Debris flow surface water level	
Bda	:	Width of the flow	

Bda= $n_{r1} \times z + n_{l1} \times z$ $(0<z<z1)$

$n_{r1} \times z + n_{l2} \times z$ $(z1<z<z2)$

$n_{r2} \times z + n_{l2} \times z$ $(z2<z)$ ･･･(1)



5.1.2 debris flow depth (Dd)

Dd= Ad ／ Bda ･･･(2)

Ad : Cross-sectional aera of debris flow peal

Ad= Ad1 $(0<z<z1)$

Ad2 $(z1<z<z2)$

Ad3 $(z2<z)$

5.1.3 Debris flow velocity (U)

$$U = \frac{1}{Kn} \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2} \quad \cdots(3)$$

Kn	:	Roughness coefficient	0.10		
θ	:	Riverbed surface slope	1.72	(degrees)	(i=1／33.3)

5.1.4 Calculate debris flow velocity and depth

$Q_{special}$ = U ・ Ad ･･･(4)

From formulas (1), (2), (3), and (4), when z is calculated when Qspecial matches Qsp, z is as follows.

z= 2.39m

Calculating debris flow velocity (U) and depth (Dd) from the result of z above, U and Dd are as follows.

Dd= 2.34m , U= 3.05m/s

5.2 Unit weight of debris flow (γ d)

$\gamma d = (\sigma \times Cd) + \{\rho \times (1 - Cd)\}$

$= \{(2{,}600 \times 0.30) + (1{,}200 \times (1- 0.30)\} \times 9.81/1000$

$= 15.89$ (kN/m^3)

5.3 Drag force of debris flow (F)

$$F = \alpha \cdot \frac{\gamma_d}{g} \cdot Dd \cdot U^2$$

$$= 1.0 \times \frac{15.89}{9.81} \times 2.34 \times 3.05^2$$

$= 35.26$ (KN／m)

5.4 Unit weight of debris flow in water (ρ di)

$\rho di = \gamma d - Wo$

$= 15.89 - 11.77 = 4.12$ (kN/m3)

5.5 Unit weight of sediment in water (γ s)

$\gamma s = C*(\sigma - \rho) \times g/1000$

$= 0.6 \times (2{,}600 - 1{,}200) \times 9.81/1000 = 8.24$ (kN/m3)

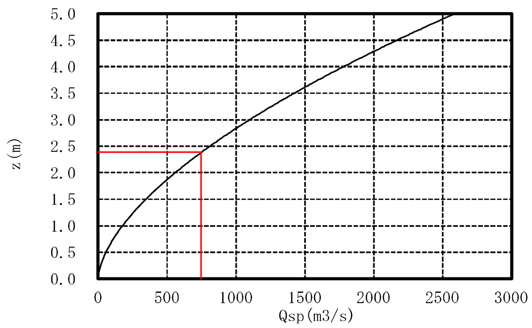
5. 6 Debris flow specifications over Riverbed

Debris flow discharge (Qsp) 742.0 m^3/s
Riverbed surface slope 1/33.3

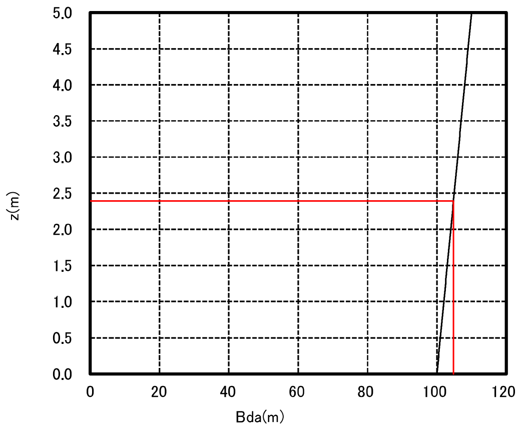
$$Q = 1/n \ast (Dd)^{2/3} \ast I^{1/2} \ast Ad$$

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
2.14	100.0	104.30	218.58	2.10	0.1	1.72	2.84	620.7
2.15	100.0	104.30	219.62	2.11	0.1	1.72	2.85	625.9
2.16	100.0	104.30	220.67	2.12	0.1	1.72	2.86	631.1
2.17	100.0	104.30	221.71	2.12	0.1	1.72	2.86	634.0
2.18	100.0	104.40	222.75	2.13	0.1	1.72	2.87	639.2
2.19	100.0	104.40	223.80	2.14	0.1	1.72	2.88	644.5
2.20	100.0	104.40	224.84	2.15	0.1	1.72	2.89	649.7
2.21	100.0	104.40	225.88	2.16	0.1	1.72	2.89	652.7
2.22	100.0	104.40	226.93	2.17	0.1	1.72	2.90	658.0
2.23	100.0	104.50	227.97	2.18	0.1	1.72	2.91	663.3
2.24	100.0	104.50	229.02	2.19	0.1	1.72	2.92	668.7
2.25	100.0	104.50	230.06	2.20	0.1	1.72	2.93	674.0
2.26	100.0	104.50	231.11	2.21	0.1	1.72	2.94	679.4
2.27	100.0	104.50	232.15	2.22	0.1	1.72	2.95	684.8
2.28	100.0	104.60	233.20	2.23	0.1	1.72	2.96	690.2
2.29	100.0	104.60	234.24	2.24	0.1	1.72	2.97	695.6
2.30	100.0	104.60	235.29	2.25	0.1	1.72	2.97	698.8
2.31	100.0	104.60	236.34	2.26	0.1	1.72	2.98	704.2
2.32	100.0	104.60	237.38	2.27	0.1	1.72	2.99	709.7
2.33	100.0	104.70	238.43	2.28	0.1	1.72	3.00	715.2
2.34	100.0	104.70	239.48	2.29	0.1	1.72	3.01	720.8
2.35	100.0	104.70	240.52	2.30	0.1	1.72	3.02	726.3
2.36	100.0	104.70	241.57	2.31	0.1	1.72	3.03	731.9
2.37	100.0	104.70	242.62	2.32	0.1	1.72	3.04	737.5
2.38	100.0	104.80	243.66	2.33	0.1	1.72	3.04	740.7
2.39	100.0	104.80	244.71	2.34	0.1	1.72	3.05	746.3
2.40	100.0	104.80	245.76	2.35	0.1	1.72	3.06	752.0
2.41	100.0	104.80	246.81	2.35	0.1	1.72	3.06	755.2
2.42	100.0	104.80	247.86	2.36	0.1	1.72	3.07	760.9
2.43	100.0	104.90	248.90	2.37	0.1	1.72	3.08	766.6
2.44	100.0	104.90	249.95	2.38	0.1	1.72	3.09	772.3
2.45	100.0	104.90	251.00	2.39	0.1	1.72	3.10	778.1
2.46	100.0	104.90	252.05	2.40	0.1	1.72	3.11	783.8
2.47	100.0	104.90	253.10	2.41	0.1	1.72	3.11	787.1
2.48	100.0	105.00	254.15	2.42	0.1	1.72	3.12	792.9
2.49	100.0	105.00	255.20	2.43	0.1	1.72	3.13	798.7
2.50	100.0	105.00	256.25	2.44	0.1	1.72	3.14	804.6
2.51	100.0	105.00	257.30	2.45	0.1	1.72	3.15	810.4
2.52	100.0	105.00	258.35	2.46	0.1	1.72	3.16	816.3
2.53	100.0	105.10	259.40	2.47	0.1	1.72	3.17	822.2
2.54	100.0	105.10	260.45	2.48	0.1	1.72	3.17	825.6
2.55	100.0	105.10	261.50	2.49	0.1	1.72	3.18	831.5
2.56	100.0	105.10	262.55	2.50	0.1	1.72	3.19	837.5
2.57	100.0	105.10	263.60	2.51	0.1	1.72	3.20	843.5
2.58	100.0	105.20	264.66	2.52	0.1	1.72	3.21	849.5
2.59	100.0	105.20	265.71	2.53	0.1	1.72	3.22	855.5
2.60	100.0	105.20	266.76	2.54	0.1	1.72	3.23	861.6
2.61	100.0	105.20	267.81	2.55	0.1	1.72	3.23	865.0
2.62	100.0	105.20	268.86	2.55	0.1	1.72	3.23	868.4
2.63	100.0	105.30	269.92	2.56	0.1	1.72	3.24	874.5
2.64	100.0	105.30	270.97	2.57	0.1	1.72	3.25	880.6

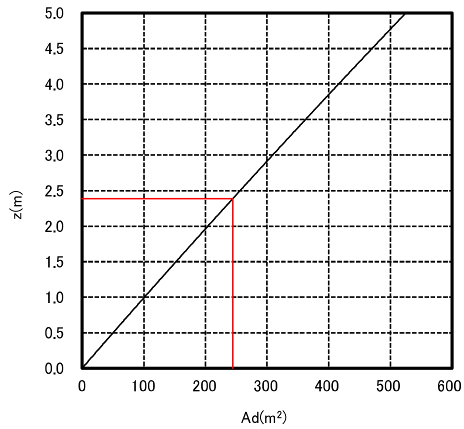
2.39	100.0	104.78	244.71	2.34	0.1	1.72	3.05	746.3



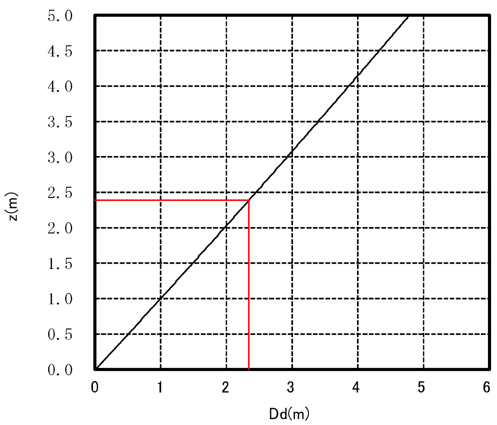
Q_{sp}-Z Curve



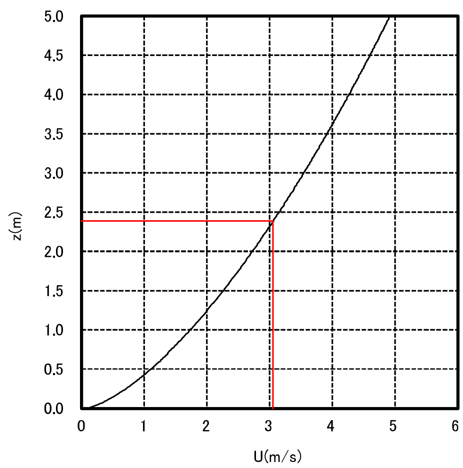
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

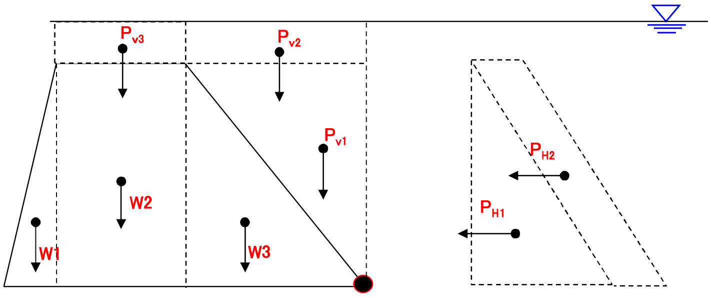
6. Stability analysis

6.1 Result of Stability analysis (overflow section)

During flooding					
Stability against overturning	e	1.092	≦	1.317	OK
Stability against sliding	n	1.33	≧	1.2	OK
Maximum ground reaction force	σ max	254.4	≦	600	OK
Minimum ground reaction force	σ min	23.8	≧	0	OK
During debris flow					
Stability against overturning	e	0.515	≦	1.317	OK
Stability against sliding	n	1.77	≧	1.2	OK
Maximum ground reaction force	σ max	172.6	≦	600	OK
Minimum ground reaction force	σ min	75.6	≧	0	OK

6.2 Verification of stability of main body (overflow section)

6.2.1 During flooding



Schematic diagram of design external forces

Remarks : Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 1.40 × 7.00 × 20.00 7.90 − 2/3 × 1.40	98.000		6.967	682.733
W2	3.00 × 7.00 × 20.00 3.50 + 1/2 × 3.00	420.000		5.000	2,100.000
W3	1/2 × 3.50 × 7.00 × 20.00 2/3 × 3.50	245.000		2.333	571.667
Hydrostatic pressure P_{V1}	1/2 × 3.50 × 7.00 × 11.77 1/3 × 3.50	144.183		1.167	168.213
P_{V2}	3.50 × 2.50 × 11.77 1/2 × 3.50	102.988		1.750	180.228
P_{V3}	3.00 × 2.50 × 11.77 3.50 + 1/2 × 3.00	88.275		5.000	441.375
P_{H1}	1/2 × 7.00 × 7.00 × 11.77 1/3 × 7.00		288.365	2.333	672.852
P_{H2}	2.50 × 7.00 × 11.77 1/2 × 7.00		205.975	3.500	720.913
TOTAL		1,098.445	494.340		5,537.981

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{5537.981}{1098.445} = 5.042$ m

Eccentric distance $e = d - B/2 = 5.042 - 7.900 / 2 = 1.092$ m

$e = 1.092$ m $\leqq B/6 = 1.317$ m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 1098.445 + 0.0 \times 7.900}{494.340} = 1.33$$

$n = 1.33 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma\, min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{1098.45}{7.90} \times \left(1 - \frac{6 \times 1.09}{7.90}\right) = 23.8 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

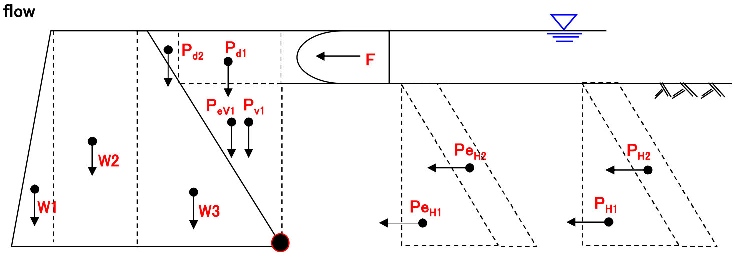
Maximum ground reaction force

$$\sigma\, max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{1098.45}{7.90} \times \left(1 + \frac{6 \times 1.09}{7.90}\right) = 254.4 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

OK

6.2.2 During bebris flow



Schematic diagram of design external forces

Remarks : Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN･m)
Self-weight of dam W1	1/2 × 1.40 × 7.00 × 20.00 7.90 − 2/3 × 1.40	98.000		6.967	682.733
W2	3.00 × 7.00 × 20.00 3.50 + 1/2 × 3.00	420.000		5.000	2,100.000
W3	1/2 × 3.50 × 7.00 × 20.00 2/3 × 3.50	245.000		2.333	571.667
Hydrostatic pressure P_{V1}	1/2 × 2.33 × 4.66 × 11.77 1/3 × 2.33	63.898		0.777	49.628
P_{H1}	1/2 × 4.66 × 4.66 × 11.77 1/3 × 4.66		127.796	1.553	198.510
P_{H2}	2.34 × 4.66 × 11.77 1/2 × 4.66		128.345	2.330	299.043
Earth pressure P_{ev1}	1/2 × 2.33 × 4.66 × 8.24 1/3 × 2.33	44.734		0.777	34.744
P_{eH1}	1/2 × 0.30 × 4.66^2 × 8.24 1/3 × 4.66		26.840	1.553	41.692
P_{eH2}	0.30 × 2.34 × 4.66 × 4.12 1/2 × 4.66		13.480	2.330	31.408
Mass of debris flow Pd1	2.33 × 2.34 × 15.89 1/2 × 2.33	86.635		1.165	100.930
Pd2	1/2 × 1.17 × 2.34 × 15.89 3.50 − 2/3 × 1.17	21.752		2.720	59.165
Drag force of debris flow F	1.00 × 15.89 × 3.05^2 × 2.34 /9.81 4.66 + 1/2 × 2.34		35.259	5.830	205.560
TOTAL		980.019	331.721		4,375.080

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{4375.080}{980.019} = 4.464$ m

Eccentric distance $e = d - B/2 = 4.464 - 7.900 / 2 = 0.515$ m

$e = 0.515$ m $\leqq$ $B/6 = 1.317$ m ……… OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 980.019 + 0.0 \times 7.90}{331.721} = 1.77$$

$n = 1.77 \geqq 1.2$ ……… OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{980.02}{7.90} \times \left(1 - \frac{6 \times 0.52}{7.90}\right) = 75.6 \ \mathrm{kN/m^2} \geqq 0 \ \mathrm{kN/m^2}$$

……… OK

Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{980.02}{7.90} \times \left(1 + \frac{6 \times 0.52}{7.90}\right) = 172.6 \ \mathrm{kN/m^2} \leqq 600 \ \mathrm{kN/m^2}$$

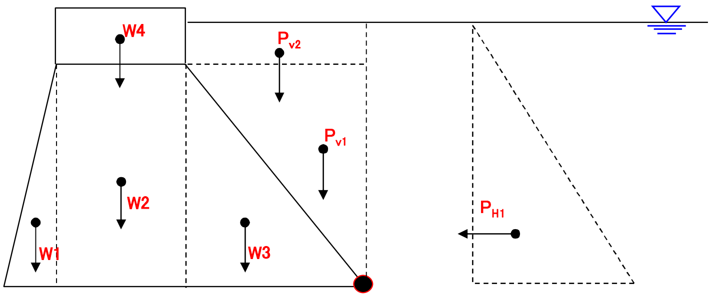
……… OK

6.3 Result of Stability calculation (non-overflow section)

During flooding					
Stability against overturning	e	1.311	≦	1.317	OK
Stability against sliding	n	1.44	≧	1.2	OK
Maximum ground reaction force	σ max	323.4	≦	600	OK
Minimum ground reaction force	σ min	0.7	≧	0	OK
During debris flow					
Stability against overturning	e	1.068	≦	1.317	OK
Stability against sliding	n	1.41	≧	1.2	OK
Maximum ground reaction force	σ max	322.9	≦	600	OK
Minimum ground reaction force	σ min	33.7	≧	0	OK

6.4 Verification of stability of main body (non-overflow section)

6.4.1 During flooding



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 1.40 × 7.00 × 20.00 7.90 − 2/3 × 1.40	98.000		6.967	682.733
W2	3.00 × 7.00 × 20.00 3.50 + 1/2 × 3.00	420.000		5.000	2,100.000
W3	1/2 × 3.50 × 7.00 × 20.00 2/3 × 3.50	245.000		2.333	571.667
W4	4.50 × 3.00 × 20.00 3.50 + 3.00 × 1/2	270.000		5.000	1,350.000
Hydrostatic pressure P_{V1}	1/2 × 3.50 × 7.00 × 11.77 1/3 × 3.50	144.183		1.167	168.213
P_{V2}	3.50 × 2.50 × 11.77 1/2 × 3.50	102.988		1.750	180.228
P_{H1}	1/2 × 9.50 × 9.50 × 11.77 1/3 × 9.50		531.121	3.167	1,681.884
TOTAL		1,280.170	531.121		6,734.725

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{6734.725}{1280.170} = 5.261$ m

Eccentric distance

$e = d - B/2 = 5.261 - 7.900 / 2 = 1.311$ m

$e = 1.311$ m $\leqq B/6 = 1.317$ m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 1280.170 + 0.0 \times 7.900}{531.121} = 1.44$$

$n = 1.44 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma\,min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{1280.17}{7.90} \times \left(1 - \frac{6 \times 1.31}{7.90}\right) = 0.7 \ \mathrm{kN/m^2} \geqq 0 \ \mathrm{kN/m^2}$$

OK

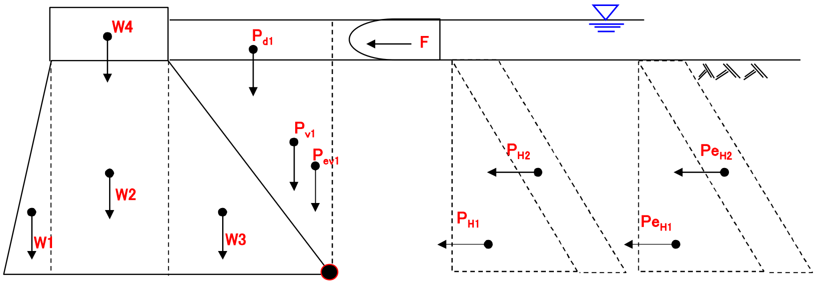
Maximum ground reaction force

$$\sigma\,max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{1280.17}{7.90} \times \left(1 + \frac{6 \times 1.31}{7.90}\right) = 323.4 \ \mathrm{kN/m^2} \leqq 600 \ \mathrm{kN/m^2}$$

OK

6.4.2 During bebris flow



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 1.40 × 7.00 × 20.00 7.90 − 2/3 × 1.40	98.000		6.967	682.733
W2	3.00 × 7.00 × 20.00 3.50 + 1/2 × 3.00	420.000		5.000	2,100.000
W3	1/2 × 3.50 × 7.00 × 20.00 2/3 × 3.50	245.000		2.333	571.667
W4	4.50 × 3.00 × 20.00 3.50 + 3.00 × 1/2	270.000		5.000	1,350.000
Hydrostatic pressure P_{V1}	1/2 × 3.50 × 7.00 × 11.77 1/3 × 3.50	144.183		1.167	168.213
P_{H1}	1/2 × 7.00 × 7.00 × 11.77 1/3 × 7.00		288.365	2.333	672.852
P_{H2}	2.34 × 7.00 × 11.77 1/2 × 7.00		192.793	3.500	674.774
Earth pressure P_{ev1}	1/2 × 3.50 × 7.00 × 8.24 1/3 × 3.50	100.940		1.167	117.763
P_{eH1}	1/2 × 0.30 × 7.00^2 × 8.24 1/3 × 7.00		60.564	2.333	141.316
P_{eH2}	0.30 × 2.34 × 7.00 × 4.12 1/2 × 7.00		20.250	3.500	70.875
Mass of debris flow Pd1	3.50 × 2.34 × 15.89 1/2 × 3.50	130.139		1.750	227.743
Drag force of debris flow F	1.00 × 15.89 × 3.05^2 × 2.34 /9.81 7.00 + 1/2 × 2.34		35.259	8.170	288.066
TOTAL		1,408.262	597.231		7,066.002

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{7066.002}{1408.262} = 5.018$ m

Eccentric distance

$e = d - B/2 = 5.018 - 7.900 / 2 = 1.068$ m

$e = 1.068$ m $\leqq B/6 = 1.317$ m ……… OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 1408.262 + 0.0 \times 7.900}{597.231} = 1.41$$

$n = 1.41 \geqq 1.2$ ……… OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{1408.26}{7.90} \times \left(1 - \frac{6 \times 1.07}{7.90}\right) = 33.7 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

……… OK

Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{1408.26}{7.90} \times \left(1 + \frac{6 \times 1.07}{7.90}\right) = 322.9 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

……… OK

7. Downstream riverbed protection works※1

7.1 Thickness of the apron

Thickness of the apron is calculated as follow.

Empirical formula (If there is the water cushion)

$t= 0.2 \times (0.6H_1 + 3h_3 -1.0)$

Where,

t : Thickness of the apron (m)

H_1 : Height from the top of the apron to the base of the spillway (m)

h_3 : Design water depth(m)

If H : Dam height(m)

Then $H_1 = H-t$

$t= 0.2 \times (0.6(H-t) + 3h_3 -1.0)$

$= -0.12t+(0.2 \times (0.6H+ 3h_3-1.0)$

Therefore,

$t= 0.2 \times (0.6H + 3h_3 -1.0)/ 1.12$

In case of the planned main dam,

Then H = 7.0 m

h_3 = 2.5 m

$t= 0.2 \times (0.6 \times 7.0 + 3 \times 2.5-1.0)/ 1.12$

$=$ 1.91071 m

$t=$ 2.0 m

⇒ Adopted value : 2.0 m

7.2 Overlap height

Length of the apron (L) is calculated as follow.

$L=(1.5\sim2.0) \times (H_1+H_3)$

L=1.5×(H1＋H3)＝ 11.3 m

L=2.0×(H1＋H3)＝ 15.0 m

⇒ Adopted value : 25.0 m

8. Ability to flow down of the opening section

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Number of opening section		Parts	14
Peak discharge of water	Q	m^3/s	371
Target discharge amount per part		$m^3/s/part$	26.50
Coefficient	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Width of opening section	B1	m	3.0

The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Q= \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1+2B_2) \times Dh^{3/2}$$

Where, Dh : Water depth (m)

B_2 : Width of overflow (m)

In case of Dh=2.9

$$Q= \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 3.0+2 \times 3.0) \times 2.9\hat{}(3/2)= \quad 26.24 \quad < 26.50$$

・・・・・NG

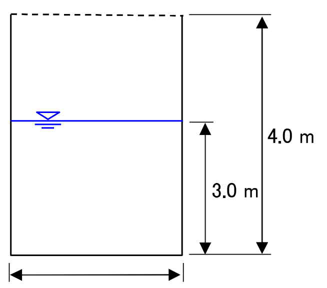
In case of Dh=3.0

$$Q= \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 3.0+2 \times 3.0) \times 3.0\hat{}(3/2)= \quad 27.61 \quad \geqq 26.50$$

・・・・・OK

Therefore,

Dh = 3.0 m



Schematic diagram of opening section

The overflow water depth is 3m, and the opening height is 4m, so the target water flow can flow down only through the opening section.

DD Leprak 3

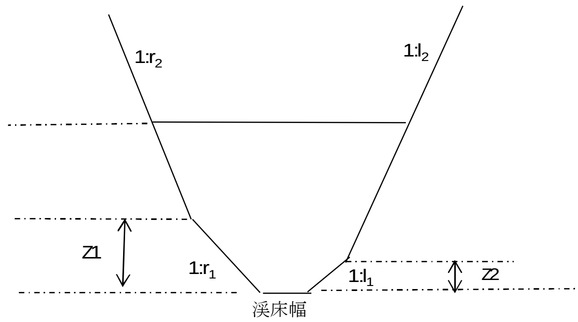
Stability calculation of S4-3 DD Leprak3
(Dam height H＜15m)

1. Design specifications

1. 1 Design condition list

①Topography, geology, facility shape conditions

Symbol	Item	Unit	Value
	Topographic condition		
	Width of Riverbed	m	100.0
$1:r_1$	Flow section right bank slope (bottom)		1.0
$1:r_2$	Flow section right bank slope (top)		1.0
$1:l_1$	Flow section left bank slope (bottom)		1.0
$1:l_2$	Flow section left bank slope (top)		1.0
θ	Riverbed surface slope		1/33.3
	Flow rate conditions		
A	Target basin area	km^2	25.80
P_{24}	24-hour(day) rainfall	mm	251.0
	Facility shape conditions		
H	Dam height	m	5.0
B	Crest width	m	3.0
b1	Road width	m	0.0
b2	Wing crest width	m	3.0
m	Upstream slope(overflow section)		0.40
m	Upstream slope(non-overflow section)		0.40
n	Downstream slope		0.20
B1	Base width of spillway	m	795.0
	Wing edge		0.5
	Sediment conditions		
d_{95}	Maximum boulder size	m	2.5
	Ground conditions		
τ_0	Shear strength		0
f	Friction coefficient		0.60
qu	Allowable bearing capacity	kN/m^2	600
n	Safety factor against sliding		1.2



②Design constant

We	Apparent unit weight of concrete	kN/m^3	20
We	Unit weight of concrete	kN/m^3	22.6
Wo	Unit weight of water	kN/m^3	11.77
σ	Gravel density	kg/m^3	2,600
ρ	Water density	kg/m^3	1,200
K_{p1}	Coefficient		120
C	Coefficient of overflow		0.6
g	Gravitational acceleration	m/s^2	9.81
C_*	metric concentration of deposited sediment		0.6
Kn	Roughness coefficient(front part)		0.1
C	Coefficient of earth pressure		0.3
ϕ	Internal friction angle of sediment	degree	35.0
τ c	Shear strength of concrete		2,760

Regarding the shear strength, the smaller value in between the dambody and the ground is used in calculation.

1.2 Apparent unit weight of concrete ※1

Symbol	Item	Unit	Value
B1	Base width of spillway	m	795.0
b	Width of opening section	m	3.0
h	Height of opening section	m	2.0
h1	Slab above the opening	m	0.0
	Number of openings	parts	38
W	Dam body volume except opening	m3	5,142
Vc	Dam body volume including opening	m3	5,963
Wc	Unit weight of concrete	kN/m3	22.6
Wγc	Apparent unit weight of concrete	kN/m3	19.49

The apparent unit weight of concrete adopts 20.0 kN/m3

⇒ Adopted value : 20.0 kN/m3

3) If the permeable part is made of concrete, the self-weight of the dam body is calculated using the dam body block volume (V_c) which is calculated by assuming the overflow section as a closed structure, and the dam body block weight (W_{rc}) which is calculated by assuming the overflow section as an open structure (Figure 11). Further, it is noted that the dam body block refers to the concrete block in the permeable part, not the concrete block used at the time of casting.

$$\gamma_{rc} = W_{rc} / V_C \quad \cdots\cdots (6)$$

Where,

γ_{rc} : apparent unit weight of concrete (kN/m^3),

W_{rc} : dam body block weight (kN) calculated by assuming the overflow section as an open structure with concrete,

V_c : dam body block volume (m^3) calculated by assuming the overflow section as a closed structure.

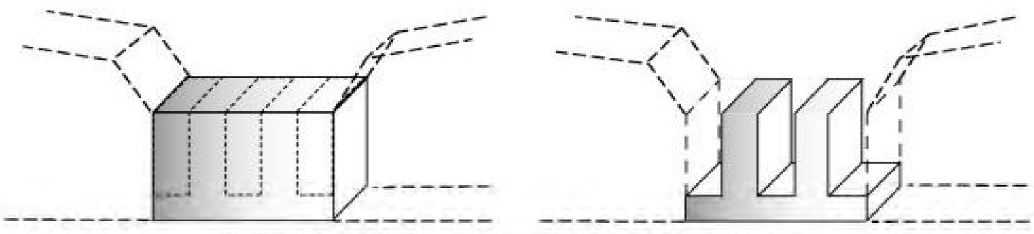


Figure 11 Dam body volume of spillway of the slit part

4) Combinations of design external forces excluding the self-weight of dam body are as shown in Table 4.

※1 Source : Technical Guideline for Designing Sabo Facilities

2. Calculation of flow discharge considering sediment content (1.5Qp)

2.1 Flow discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Target basin area	A	km^2	25.80

The flow discharge considering sediment content is calculated by under rainfall(251mm/24h) with the design exceedance probability.
The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Qp = \frac{1}{3.6} \times K_{f1} \times Pa \times A$$ ※1

Qp : Peak discharge of water without sediment(m^3/s)
Pa : Average rainfall intensity during the flood concentration time(mm/hr)
A : Target basin area(km^2)
K_{f1} : Coefficient

Therefore, the peak discharge of water without sediment is as follow.

Qp＝ 371 (m^3/s)

Flow discharge considering sediment content is 1.5 times the peak discharge of water without sediment, and calculated as follow.

Q= 1.5 × Qp ※1
= 1.5 × 371

= 556.5 (m^3/s)

※1 Source : Technical Guideline for Establishing Sabo Master Plan

2.2 Value of overflow depth in regards to the discharge bulked with sediment

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Discharge of water considering sediment content	Q	m^3/s	556.50
Coefficient of overflow	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Base width of the spillway	B1	m	795.0
Wing edge slope	m		0.5

The overflow depth in regards to discharge of water containing fine sediments is calculated as follow.

$$Q = \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1 + 2B_2) \times Dh^{3/2}$$ ※1

Where, Dh : Overflow depth (m)
B_2 : Width of overflow (m)

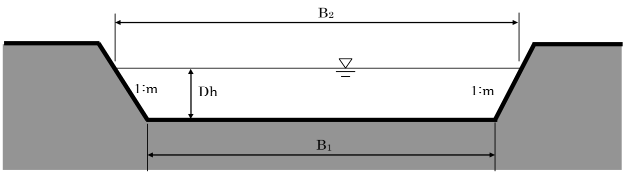
In case of Dh=0.5

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 795.0 + 2 \times 795.5) \times 0.5\hat{}(3/2)\text{:} \quad 498.12 \quad < 556.50$$

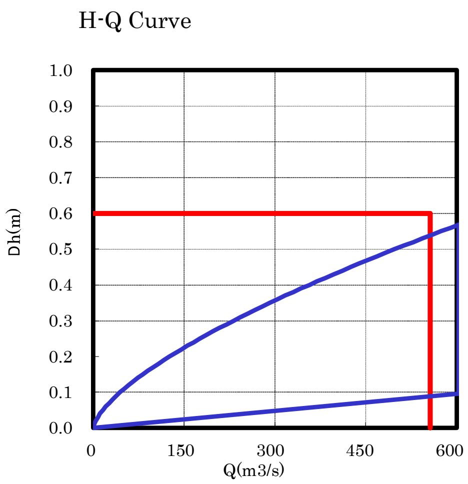
In case of Dh=0.6 ・・・・・NG

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 795.0 + 2 \times 795.6) \times 0.6\hat{}(3/2)\text{:} \quad 654.83 \quad \geqq 556.50$$

Therefore, ・・・・・OK
Dh ＝ 0.6 m



Schematic diagram of the cross-sectional shape of Spillway



※1 Source : SNI 2851-2021_Desain Saboda

3. Calculation of debris flow peak discharge (Qsp)

3.1 Debris flow peak discharge (Qsp) ※1

Specifications list

Symbol	Item	Unit	Value
σ	Density of gravel	kg/m^3	2,600
ρ	Density of water	kg/m^3	1,200
Wo	Unit weight of the water	kN/m^3	11.77
ϕ	Internal friction angle of the deposited soil	degrees	35.0
d	Maximum boulder size	m	2.50
$\tan\theta$	Riverbed surface slope		1/33.3
C_*	Volumetric concentration of deposited sediment		0.60
Kn	Roughness coefficient (the front part of debris flow)		0.10
B	Width of riverbed	m	100.0

3.1.1 Concentration of debris flow (Takahashi's Equation)

$$Cd = \frac{\rho \cdot \tan\theta}{(\sigma-\rho)\cdot(\tan\phi - \tan\theta)}$$

Where,

ρ	;	Density of water	1,200	(Kg/m^3)
θ	;	Riverbed surface slope	1.72	(degrees) (i=1/33.3)
σ	;	Density of gravel	2,600	(Kg/m^3)
ϕ	;	Internal friction angle of sediment deposited on riverbed	35.0	(degrees)

$$= \frac{1{,}200 \times \tan 1.72}{(2{,}600 - 1{,}200) \times (\tan 35.0 - \tan 1.72)}$$

$$= 0.04$$

When the calculated value (Cd) is larger than 0.9C*, , Cd is assumed to be equal to 0.9C*. Meanwhile, if the calculated value (Cd) is smaller than 0.3, Cd value is assumed to be 0.3

$\therefore$ Cd = 0.30

3.1.2 Debris flow peak discharge (Qsp)

The peak discharge of debris flow Qsp (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Qp (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp$$

Where,

Cd	:	Concentration of debris flow	=	0.30
C*	:	Volumetric concentration of deposited sediment	=	0.60
Qp	:	Peak discharge of water	=	371.00 (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp = \frac{0.60}{0.60 - 0.30} \times 371 = 742.0$$

Qsp= 742.0 (m^3/sec)

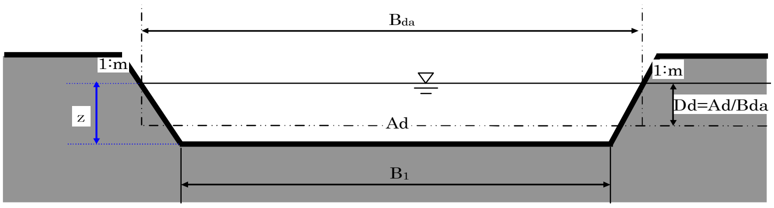
※1 Source : SNI 2851-2021_Desain Sabodam

3.2 Value of the overflow depth for debris flow peak discharge (Qsp).

3.2.1 Calculation of overflow depth ※1

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Debris flow peak discharge	Qsp	m^3/s	742.0
Roughness coefficient	Kn		0.1
Design deposit surface slope	θ	degrees	1.43
Base width of spillway	B1	m	795.0
Wing edge slope	m		0.5



The amount of the debris flow that can flow down through this cross section is determined as follow.

$$Qsp=U \cdot Ad$$

Where,

z :	Debris flow surface water level	(m)
U :	Debris flow velocity	(m/s)
Bda :	Width of the flow	(m)
Ad :	Cross−sectional aera of debris flow peak discharge	(m^2)
Dd :	Debris flow depth	(m)

U, Bda, Ad and Dd can be calculated as follow.

$$U= 1/Kn \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2}$$

$$Dd= Ad/Bda$$

$$Ad= (B1+m \times z) \times z$$

$$Bda= B1+2 \times m \times z$$

If the debris flow depth is 0.72m

Qsp= 1.27 × 572.66

= 727.2 < 742.0 ・・・・・NG

U= 1/0.1 × $0.72^{2/3}$ × $0.02^{1/2}$ = 1.27

Dd= 572.66/ 795.72 = 0.72

Ad= (795.0 +0.5 × 0.72) × 0.72 = 572.66

Bda= 795.0 +2 × 0.50 × 0.72 = 795.72

If the debris flow depth is 0.73m

Qsp= 1.28 × 580.62

= 743.1 ≧ 742.0 ・・・・・OK

U= 1/0.1 × $0.73^{2/3}$ × $0.02^{1/2}$ = 1.28

Dd= 580.62/ 795.73 = 0.73

Ad= (795.0 +0.5 × 0.73) × 0.73 = 580.62

Bda= 795.0 +2 × 0.50 × 0.73 = 795.73

Therefore, the debris flow depth is adapted as 0.8m

※1 Source : Technical Guideline for Establishing Sabo Master Plan

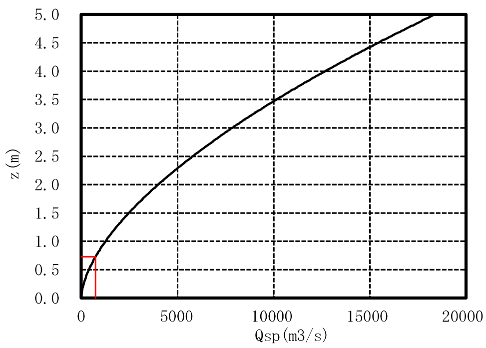
3.2.2 Debris flow specifications over spillway

$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

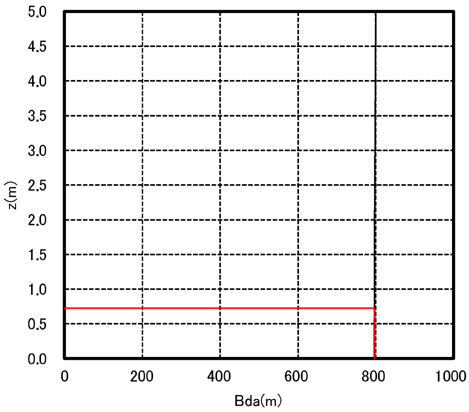
Debris flow discharge (Qsp) 742.0 m^3/s
Design deposit surface slope 1/40.0

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
0.48	795.0	795.47	373.76	0.47	0.1	1.43	1.0	355.0
0.49	795.0	795.48	381.72	0.48	0.1	1.43	1.0	370.2
0.50	795.0	795.49	389.67	0.49	0.1	1.43	1.0	381.8
0.51	795.0	795.50	397.63	0.50	0.1	1.43	1.0	397.6
0.52	795.0	795.51	405.58	0.51	0.1	1.43	1.0	409.6
0.53	795.0	795.52	413.54	0.52	0.1	1.43	1.0	421.8
0.54	795.0	795.53	421.49	0.53	0.1	1.43	1.0	434.1
0.55	795.0	795.54	429.45	0.54	0.1	1.43	1.1	450.9
0.56	795.0	795.55	437.40	0.55	0.1	1.43	1.1	463.6
0.57	795.0	795.56	445.36	0.56	0.1	1.43	1.1	476.5
0.58	795.0	795.57	453.31	0.57	0.1	1.43	1.1	494.1
0.59	795.0	795.58	461.27	0.58	0.1	1.43	1.1	507.3
0.60	795.0	795.59	469.22	0.59	0.1	1.43	1.1	520.8
0.61	795.0	795.60	477.18	0.60	0.1	1.43	1.1	534.4
0.62	795.0	795.61	485.14	0.61	0.1	1.43	1.1	553.0
0.63	795.0	795.62	493.09	0.62	0.1	1.43	1.2	567.0
0.64	795.0	795.63	501.05	0.63	0.1	1.43	1.2	581.2
0.65	795.0	795.64	509.00	0.64	0.1	1.43	1.2	595.5
0.66	795.0	795.65	516.96	0.65	0.1	1.43	1.2	615.1
0.67	795.0	795.66	524.92	0.66	0.1	1.43	1.2	629.9
0.68	795.0	795.67	532.87	0.67	0.1	1.43	1.2	644.7
0.69	795.0	795.69	548.79	0.69	0.1	1.43	1.2	675.0
0.70	795.0	795.70	556.75	0.70	0.1	1.43	1.3	695.9
0.71	795.0	795.71	564.70	0.71	0.1	1.43	1.3	711.5
0.72	795.0	795.72	572.66	0.72	0.1	1.43	1.3	727.2
0.73	795.0	795.73	580.62	0.73	0.1	1.43	1.3	743.1
0.74	795.0	795.74	588.57	0.74	0.1	1.43	1.3	759.2
0.75	795.0	795.75	596.53	0.75	0.1	1.43	1.3	775.4
0.76	795.0	795.76	604.49	0.76	0.1	1.43	1.3	797.9
0.77	795.0	795.77	612.45	0.77	0.1	1.43	1.3	814.5
0.78	795.0	795.78	620.40	0.78	0.1	1.43	1.3	831.3
0.79	795.0	795.79	628.36	0.79	0.1	1.43	1.4	848.2
0.80	795.0	795.80	636.32	0.80	0.1	1.43	1.4	865.3
0.81	795.0	795.81	644.28	0.81	0.1	1.43	1.4	882.6
0.82	795.0	795.82	652.24	0.82	0.1	1.43	1.4	900.0
0.83	795.0	795.83	660.19	0.83	0.1	1.43	1.4	924.2
0.84	795.0	795.84	668.15	0.84	0.1	1.43	1.4	942.0
0.85	795.0	795.85	676.11	0.85	0.1	1.43	1.4	960.0
0.86	795.0	795.86	684.07	0.86	0.1	1.43	1.4	978.2
0.87	795.0	795.87	692.03	0.87	0.1	1.43	1.4	996.5
0.88	795.0	795.88	699.99	0.88	0.1	1.43	1.5	1014.9
0.89	795.0	795.89	707.95	0.89	0.1	1.43	1.5	1033.6
0.90	795.0	795.90	715.91	0.90	0.1	1.43	1.5	1052.3
0.91	795.0	795.91	723.86	0.91	0.1	1.43	1.5	1071.3
0.92	795.0	795.92	731.82	0.92	0.1	1.43	1.5	1090.4
0.93	795.0	795.93	739.78	0.93	0.1	1.43	1.5	1117.0
0.94	795.0	795.94	747.74	0.94	0.1	1.43	1.5	1136.5
0.95	795.0	795.95	755.70	0.95	0.1	1.43	1.5	1156.2
0.96	795.0	795.96	763.66	0.96	0.1	1.43	1.5	1176.0
0.97	795.0	795.97	771.62	0.97	0.1	1.43	1.6	1196.0
0.98	795.0	795.98	779.58	0.98	0.1	1.43	1.6	1216.1

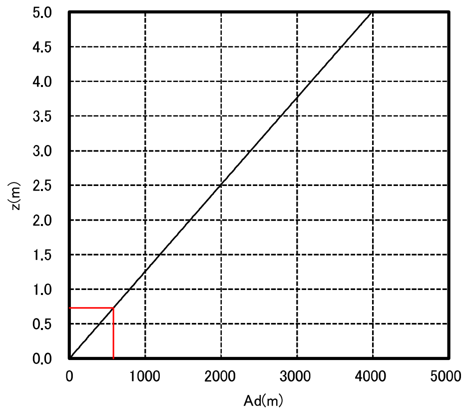
0.73	795.0	795.73	580.62	0.73	0.1	1.43	1.3	743.1



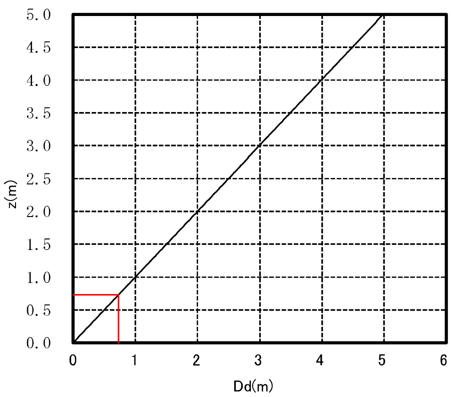
Q_{sp}-Z Curve



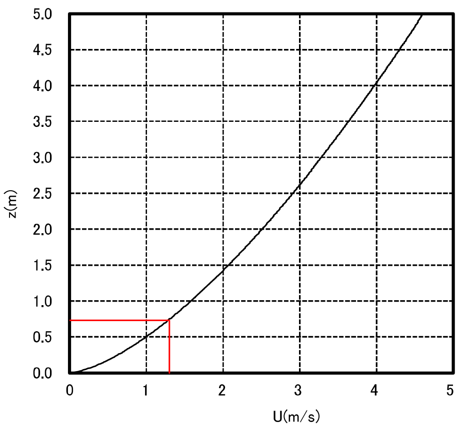
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

4. Design water depth※1

The design water depth is the largest of the values from the calculation results of below chart.
Therefore, since the Maximum boulder size has the largest value, the design water depth is determined to be 2.5m.

Table 1 Overflow water depth calculated by each method

Item	Depth(m)
Value of the overflow depth of debris flow peak discharge	0.8
Value of overflow depth of discharge considering sediment content	0.6
Maximum boulder size	2.5

The freeboard is set based on Table 2. However, the freeboard height shall be designed according to the riverbed slope and freeboard height ratio to the design water depth must not be less than the value presented in Table 3. The riverbed slope referred here is the design sediment deposition slope.

Table 2 Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6 m
200 ～ 500 m^3/s	0.8 m
500 m^3/s or more	1.0 m

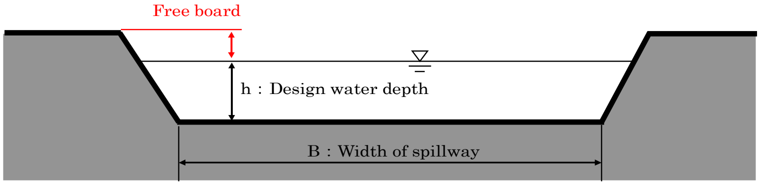


Table 3 Minimum value of the ratio of the freeboard height to the design water depth by riverbed slope

Riverbed slope	(Freeboard height)/(Design water depth)
1/10 or more	0.50
1/10～1/30	0.40
1/30～1/50	0.30
1/50～1/70	0.25

Design deposit surface slope : 1/40
Lowest value of (Freeboard height)/(Design water depth) : 0.3

Since the design discharge is 556.5m3/s, the freeboard is 1.0 m
(Freeboard height)/(Design water depth) is 1m / 2.5m = 0.40 ＞ 0.3
(Minimum required freeboard height = 0.75m)

Therefore, the freeboard height adopts 1.0m

As a result of the above, Spillway height is 2.5 + 1 = 3.5 m

However, the spillway height will be adjusted to match the height of the existing Dike, and a height of 5.0 m will be adopted.

※1 Source : SNI 2851-2021_Desain Sabodam

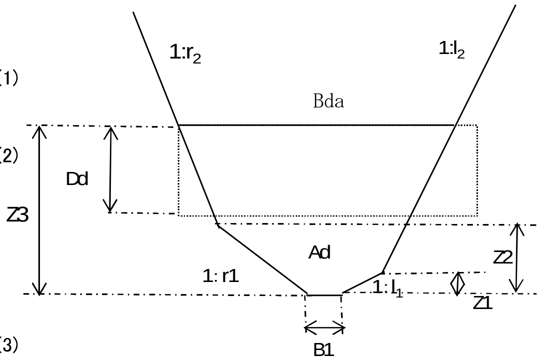
5. Specifications of debris flow

5.1 Setting of debris flow velocity and depth based on debris flow peak discharge

5.1.1 Debris flow cross-section

n_{r1} : Flow section right bank slope (bottom) 1.00
n_{r2} : Flow section right bank slope (top) 1.00
n_{l1} : Flow section left bank slope (bottom) 1.00
n_{l2} : Flow section left bank slope (top) 1.00
z : Debris flow surface water level
Bda : Width of the flow

Bda= $n_{r1} \times z + n_{l1} \times z$ (0<z<z1)
$n_{r1} \times z + n_{l2} \times z$ (z1<z<z2)
$n_{r2} \times z + n_{l2} \times z$ (z2<z) ･･･(1)



5.1.2 debris flow depth (Dd)

Dd= Ad ／ Bda ･･･(2)

Ad : Cross-sectional aera of debris flow peal

Ad= Ad1 (0<z<z1)
Ad2 (z1<z<z2)
Ad3 (z2<z)

5.1.3 Debris flow velocity (U)

$$U = \frac{1}{Kn} \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2} \quad \cdots(3)$$

Kn : Roughness coefficient 0.10
θ : Riverbed surface slope 1.72 (degrees) (i=1／33.3)

5.1.4 Calculate debris flow velocity and depth

$Q_{special}$ = U ・ Ad ･･･(4)

From formulas (1), (2), (3), and (4), when z is calculated when Qspecial matches Qsp, z is as follows.

z= 2.39m

Calculating debris flow velocity (U) and depth (Dd) from the result of z above, U and Dd are as follows.

Dd= 2.34m , U= 3.05m/s

5.2 Unit weight of debris flow (γ d)

$\gamma d = (\sigma \times Cd) + \{\rho \times (1 - Cd)\}$

$= \{(\ 2{,}600 \times 0.30\ ) + (\ 1{,}200 \times (1- 0.30\ )\} \times 9.81/1000$

$=$ 15.89 (kN/m^3)

5.3 Drag force of debris flow (F)

$$F = \alpha \cdot \frac{\gamma_d}{g} \cdot Dd \cdot U^2$$

$$= 1.0 \times \frac{15.89}{9.81} \times 2.34 \times 3.05^2$$

$=$ 35.26 (KN／m) （少数第3位切り上げ）

5.4 Unit weight of debris flow in water (ρ di)

ρ di= γ d－Wo

$=$ 15.89 － 11.77 ＝ 4.12 (kN/m3)

5.5 Unit weight of sediment in water (γ s)

γ s=C*($\sigma - \rho$) × g/1000

= 0.6 × (2,600 － 1,200) × 9.81/1000 = 8.24 (kN/m3)

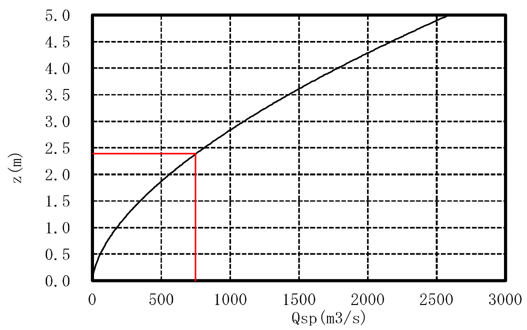
5. 6 Debris flow specifications over Riverbed

Debris flow discharge (Qsp) 742.0 m^3/s
Riverbed surface slope 1/33.3

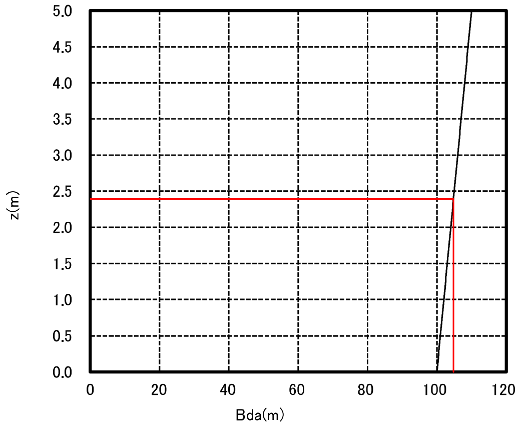
$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
2.14	100.0	104.30	218.58	2.10	0.1	1.72	2.84	620.7
2.15	100.0	104.30	219.62	2.11	0.1	1.72	2.85	625.9
2.16	100.0	104.30	220.67	2.12	0.1	1.72	2.86	631.1
2.17	100.0	104.30	221.71	2.12	0.1	1.72	2.86	634.0
2.18	100.0	104.40	222.75	2.13	0.1	1.72	2.87	639.2
2.19	100.0	104.40	223.80	2.14	0.1	1.72	2.88	644.5
2.20	100.0	104.40	224.84	2.15	0.1	1.72	2.89	649.7
2.21	100.0	104.40	225.88	2.16	0.1	1.72	2.89	652.7
2.22	100.0	104.40	226.93	2.17	0.1	1.72	2.90	658.0
2.23	100.0	104.50	227.97	2.18	0.1	1.72	2.91	663.3
2.24	100.0	104.50	229.02	2.19	0.1	1.72	2.92	668.7
2.25	100.0	104.50	230.06	2.20	0.1	1.72	2.93	674.0
2.26	100.0	104.50	231.11	2.21	0.1	1.72	2.94	679.4
2.27	100.0	104.50	232.15	2.22	0.1	1.72	2.95	684.8
2.28	100.0	104.60	233.20	2.23	0.1	1.72	2.96	690.2
2.29	100.0	104.60	234.24	2.24	0.1	1.72	2.97	695.6
2.30	100.0	104.60	235.29	2.25	0.1	1.72	2.97	698.8
2.31	100.0	104.60	236.34	2.26	0.1	1.72	2.98	704.2
2.32	100.0	104.60	237.38	2.27	0.1	1.72	2.99	709.7
2.33	100.0	104.70	238.43	2.28	0.1	1.72	3.00	715.2
2.34	100.0	104.70	239.48	2.29	0.1	1.72	3.01	720.8
2.35	100.0	104.70	240.52	2.30	0.1	1.72	3.02	726.3
2.36	100.0	104.70	241.57	2.31	0.1	1.72	3.03	731.9
2.37	100.0	104.70	242.62	2.32	0.1	1.72	3.04	737.5
2.38	100.0	104.80	243.66	2.33	0.1	1.72	3.04	740.7
2.39	100.0	104.80	244.71	2.34	0.1	1.72	3.05	746.3
2.40	100.0	104.80	245.76	2.35	0.1	1.72	3.06	752.0
2.41	100.0	104.80	246.81	2.35	0.1	1.72	3.06	755.2
2.42	100.0	104.80	247.86	2.36	0.1	1.72	3.07	760.9
2.43	100.0	104.90	248.90	2.37	0.1	1.72	3.08	766.6
2.44	100.0	104.90	249.95	2.38	0.1	1.72	3.09	772.3
2.45	100.0	104.90	251.00	2.39	0.1	1.72	3.10	778.1
2.46	100.0	104.90	252.05	2.40	0.1	1.72	3.11	783.8
2.47	100.0	104.90	253.10	2.41	0.1	1.72	3.11	787.1
2.48	100.0	105.00	254.15	2.42	0.1	1.72	3.12	792.9
2.49	100.0	105.00	255.20	2.43	0.1	1.72	3.13	798.7
2.50	100.0	105.00	256.25	2.44	0.1	1.72	3.14	804.6
2.51	100.0	105.00	257.30	2.45	0.1	1.72	3.15	810.4
2.52	100.0	105.00	258.35	2.46	0.1	1.72	3.16	816.3
2.53	100.0	105.10	259.40	2.47	0.1	1.72	3.17	822.2
2.54	100.0	105.10	260.45	2.48	0.1	1.72	3.17	825.6
2.55	100.0	105.10	261.50	2.49	0.1	1.72	3.18	831.5
2.56	100.0	105.10	262.55	2.50	0.1	1.72	3.19	837.5
2.57	100.0	105.10	263.60	2.51	0.1	1.72	3.20	843.5
2.58	100.0	105.20	264.66	2.52	0.1	1.72	3.21	849.5
2.59	100.0	105.20	265.71	2.53	0.1	1.72	3.22	855.5
2.60	100.0	105.20	266.76	2.54	0.1	1.72	3.23	861.6
2.61	100.0	105.20	267.81	2.55	0.1	1.72	3.23	865.0
2.62	100.0	105.20	268.86	2.55	0.1	1.72	3.23	868.4
2.63	100.0	105.30	269.92	2.56	0.1	1.72	3.24	874.5
2.64	100.0	105.30	270.97	2.57	0.1	1.72	3.25	880.6

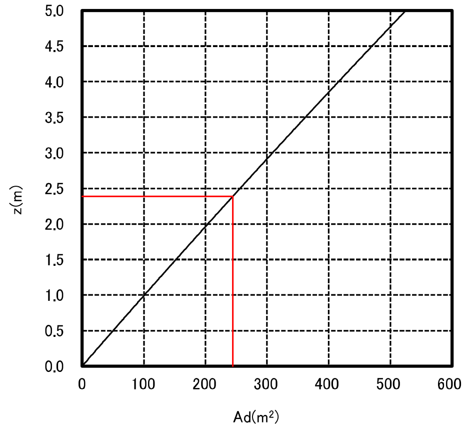
2.39	100.0	104.78	244.71	2.34	0.1	1.72	3.05	746.3



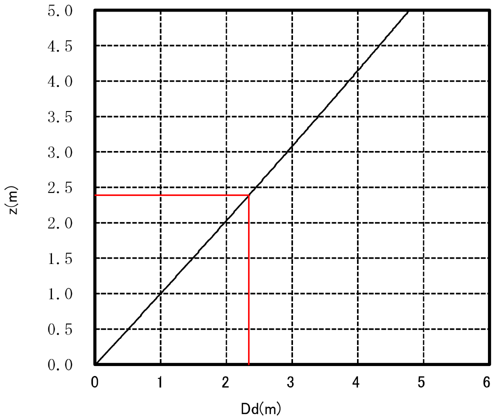
Q_{sp}-Z Curve



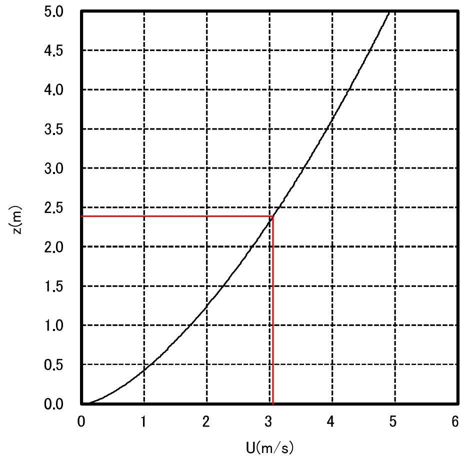
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

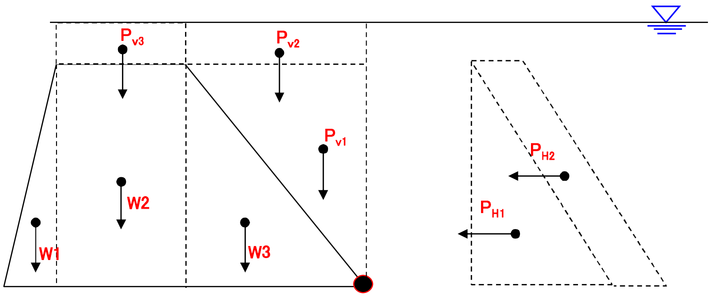
6. Stability analysis

6.1 Result of Stability analysis (overflow section)

During flooding					
Stability against overturning	e	0.766	≦	1.000	OK
Stability against sliding	n	1.33	≧	1.2	OK
Maximum ground reaction force	σ max	193.1	≦	600	OK
Minimum ground reaction force	σ min	25.6	≧	0	OK
During debris flow					
Stability against overturning	e	0.349	≦	1.000	OK
Stability against sliding	n	1.92	≧	1.2	OK
Maximum ground reaction force	σ max	120.4	≦	600	OK
Minimum ground reaction force	σ min	58.1	≧	0	OK

6.2 Verification of stability of main body (overflow section)

6.2.1 During flooding



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN･m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 6.00 − 2/3 × 1.00	50.000		5.333	266.667
W2	3.00 × 5.00 × 20.00 2.00 ＋ 1/2 × 3.00	300.000		3.500	1,050.000
W3	1/2 × 2.00 × 5.00 × 20.00 2/3 × 2.00	100.000		1.333	133.333
Hydrostatic pressure P_{V1}	1/2 × 2.00 × 5.00 × 11.77 1/3 × 2.00	58.850		0.667	39.233
P_{V2}	2.00 × 2.50 × 11.77 1/2 × 2.00	58.850		1.000	58.850
P_{V3}	3.00 × 2.50 × 11.77 2.00 ＋ 1/2 × 3.00	88.275		3.500	308.963
P_{H1}	1/2 × 5.00 × 5.00 × 11.77 1/3 × 5.00		147.125	1.667	245.208
P_{H2}	2.50 × 5.00 × 11.77 1/2 × 5.00		147.125	2.500	367.813
TOTAL		655.975	294.250		2,470.067

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{2470.067}{655.975} = 3.765$ m

Eccentric distance

$e = d - B/2 = 3.765 - 6.000 / 2 = 0.766$ m

$e = 0.766$ m $\leqq B/6 = 1.000$ m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 655.975 + 0.0 \times 6.000}{294.250} = 1.33$$

$n = 1.33 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma_{min} = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{655.98}{6.00} \times \left(1 - \frac{6 \times 0.77}{6.00}\right) = 25.6 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

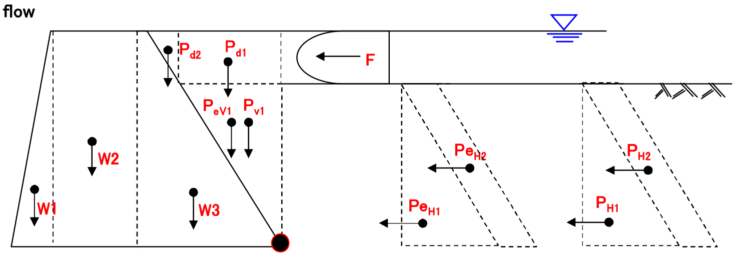
Maximum ground reaction force

$$\sigma_{max} = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{655.98}{6.00} \times \left(1 + \frac{6 \times 0.77}{6.00}\right) = 193.1 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

OK

6.2.2 During bebris flow



Schematic diagram of design external forces

Remarks：Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN･m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 6.00 − 2/3 × 1.00	50.000		5.333	266.667
W2	3.00 × 5.00 × 20.00 2.00 ＋ 1/2 × 3.00	300.000		3.500	1,050.000
W3	1/2 × 2.00 × 5.00 × 20.00 2/3 × 2.00	100.000		1.333	133.333
Hydrostatic pressure P_{V1}	1/2 × 1.06 × 2.66 × 11.77 1/3 × 1.06	16.656		0.355	5.907
P_{H1}	1/2 × 2.66 × 2.66 × 11.77 1/3 × 2.66		41.640	0.887	36.921
P_{H2}	2.34 × 2.66 × 11.77 1/2 × 2.66		73.261	1.330	97.437
Earth pressure P_{ev1}	1/2 × 1.06 × 2.66 × 8.24 1/3 × 1.06	11.661		0.355	4.136
P_{eH1}	1/2 × 0.30 × 2.66 ^{2}× 8.24 1/3 × 2.66		8.745	0.887	7.754
P_{eH2}	0.30 × 2.34 × 2.66 × 4.12 1/2 × 2.66		7.690	1.330	10.228
Mass of debris flow Pd1	1.06 × 2.34 × 15.89 1/2 × 1.06	39.562		0.532	21.047
Pd2	1/2 × 0.94 × 2.34 × 15.89 2.00 − 2/3 × 0.94	17.401		1.376	23.944
Drag force of debris flow F	1.00 × 15.89 × 3.05 ^{2} × 2.34 /9.81 2.66 ＋ 1/2 × 2.34		35.259	3.830	135.042
TOTAL		535.280	166.596		1,792.416

Stability against overturning

Resultant force position $d=\dfrac{\Sigma M}{\Sigma V}=\dfrac{1792.416}{535.280}=3.349$ m

Eccentric distance

$e= d-B/2 = 3.349 - 6.000 / 2 = 0.349$ m

$e= 0.349$ m $\leqq B/6= 1.000$ m ………… OK

Stability against sliding

$$n=\frac{f\times \Sigma V+\tau \times L}{H}$$

$$=\frac{0.600 \times 535.280 + 0.0 \times 6.00}{166.596} = 1.92$$

$n= 1.92 \geqq 1.2$

………… OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min=\frac{\Sigma V}{B}\times\left(1-\frac{6\times e}{B}\right)$$

$$=\frac{535.28}{6.00}\times\left(1-\frac{6\times 0.35}{6.00}\right)=58.1\ \mathrm{kN/m^2} \geqq 0\ \mathrm{kN/m^2}$$

………… OK

Maximum ground reaction force

$$\sigma max=\frac{\Sigma V}{B}\times\left(1+\frac{6\times e}{B}\right)$$

$$=\frac{535.28}{6.00}\times\left(1+\frac{6\times 0.35}{6.00}\right)=120.4\ \mathrm{kN/m^2} \leqq 600\ \mathrm{kN/m^2}$$

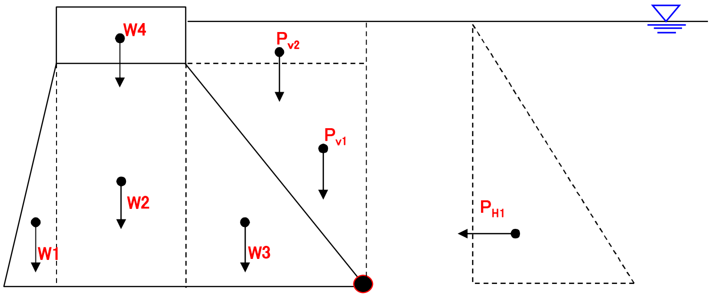
………… OK

6.3 Result of Stability analysis (non-overflow section)

During flooding					
Stability against overturning	e	0.948	≦	1.000	OK
Stability against sliding	n	1.57	≧	1.2	OK
Maximum ground reaction force	σmax	281.8	≦	600	OK
Minimum ground reaction force	σmin	7.6	≧	0	OK
During debris flow					
Stability against overturning	e	0.825	≦	1.000	OK
Stability against sliding	n	1.51	≧	1.2	OK
Maximum ground reaction force	σmax	281.2	≦	600	OK
Minimum ground reaction force	σmin	27.0	≧	0	OK

6.4 Verification of stability of main body (non-overflow section)

6.4.1 During flooding



Schematic diagram of design external forces

Remarks：Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 6.00 − 2/3 × 1.00	50.000		5.333	266.667
W2	3.00 × 5.00 × 20.00 2.00 ＋ 1/2 × 3.00	300.000		3.500	1,050.000
W3	1/2 × 2.00 × 5.00 × 20.00 2/3 × 2.00	100.000		1.333	133.333
W4	5.00 × 3.00 × 20.00 2.00 ＋ 3.00 × 1/2	300.000		3.500	1,050.000
Hydrostatic pressure P_{V1}	1/2 × 2.00 × 5.00 × 11.77 1/3 × 2.00	58.850		0.667	39.233
P_{V2}	2.00 × 2.50 × 11.77 1/2 × 2.00	58.850		1.000	58.850
P_{H1}	1/2 × 7.50 × 7.50 × 11.77 1/3 × 7.50		331.031	2.500	827.578
TOTAL		867.700	331.031		3,425.661

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{3425.661}{867.700} = 3.948$ m

Eccentric distance
e= d−B/2 = 3.948 − 6.000 / 2 = 0.948 m
e= 0.948 m ≦ B/6= 1.000 m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 867.700 + 0.0 \times 6.000}{331.031} = 1.57$$

n= 1.57 ≧ 1.2

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{867.70}{6.00} \times \left(1 - \frac{6 \times 0.95}{6.00}\right) = 7.6 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

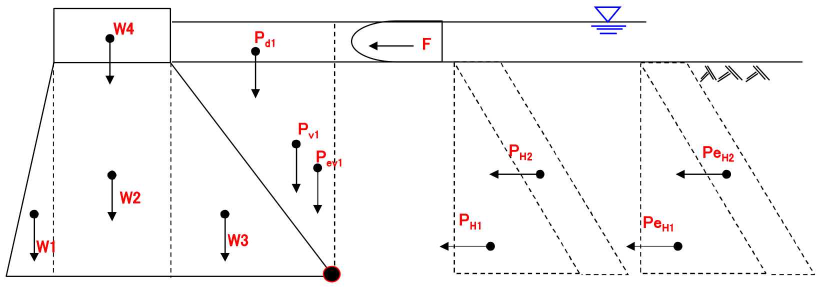
Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{867.70}{6.00} \times \left(1 + \frac{6 \times 0.95}{6.00}\right) = 281.8 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

OK

6.4.2 During bebris flow



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN･m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 6.00 − 2/3 × 1.00	50.000		5.333	266.667
W2	3.00 × 5.00 × 20.00 2.00 ＋ 1/2 × 3.00	300.000		3.500	1,050.000
W3	1/2 × 2.00 × 5.00 × 20.00 2/3 × 2.00	100.000		1.333	133.333
W4	5.00 × 3.00 × 20.00 2.00 ＋ 3.00 × 1/2	300.000		3.500	1,050.000
Hydrostatic pressure P_{V1}	1/2 × 2.00 × 5.00 × 11.77 1/3 × 2.00	58.850		0.667	39.233
P_{H1}	1/2 × 5.00 × 5.00 × 11.77 1/3 × 5.00		147.125	1.667	245.208
P_{H2}	2.34 × 5.00 × 11.77 1/2 × 5.00		137.709	2.500	344.273
Earth pressure P_{ev1}	1/2 × 2.00 × 5.00 × 8.24 1/3 × 2.00	41.200		0.667	27.467
P_{eH1}	1/2 × 0.30 × 5.00^2 × 8.24 1/3 × 5.00		30.900	1.667	51.500
P_{eH2}	0.30 × 2.34 × 5.00 × 4.12 1/2 × 5.00		14.460	2.500	36.150
Mass of debris flow Pd1	2.00 × 2.34 × 15.89 1/2 × 2.00	74.365		1.000	74.365
Drag force of debris flow F	1.00 × 15.89 × 3.05^2 × 2.34 /9.81 5.00 ＋ 1/2 × 2.34		35.259	6.170	217.548
TOTAL		924.415	365.453		3,535.744

Stability against overturning

Resultant force position $d=\dfrac{\Sigma M}{\Sigma V}=\dfrac{3535.744}{924.415}=3.825$ m

Eccentric distance $e=d-B/2=3.825-6.000/2=0.825$ m

$e=0.825$ m $\leqq B/6=1.000$ m .... OK

Stability against sliding

$$n=\frac{f\times\Sigma V+\tau\times L}{H}$$

$$=\frac{0.600\times 924.415+0.0\times 6.000}{365.453}=1.51$$

$n=1.51\geqq 1.2$ .... OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min=\frac{\Sigma V}{B}\times\left(1-\frac{6\times e}{B}\right)$$

$$=\frac{924.42}{6.00}\times\left(1-\frac{6\times 0.83}{6.00}\right)=27.0\ kN/m^2\geqq 0\ kN/m^2$$

.... OK

Maximum ground reaction force

$$\sigma max=\frac{\Sigma V}{B}\times\left(1+\frac{6\times e}{B}\right)$$

$$=\frac{924.42}{6.00}\times\left(1+\frac{6\times 0.83}{6.00}\right)=281.2\ kN/m^2\leqq 600\ kN/m^2$$

.... OK

7. Downstream riverbed protection works※1

7.1Thickness of th Length of the apron（L）is calculated as follow.

$L=(1.5\sim2.0)\times(H_1+H_3)$

L=1.5×(H1＋H3)＝ 5.7 m

L=2.0×(H1＋H3)＝ 7.6 m

Where, H_1 : Height from the top of the apron to the base of the spillway (m)

h_3 : Design water depth(m)

⇒ Adopted value : 20.0 m

8. Ability to flow down of the opening section

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Number of opening section		Parts	38
Peak discharge of water	Q	m^3/s	371
Target discharge amount per part		$m^3/s/part$	9.76
Coefficient	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Width of opening section	B1	m	3.0

The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Q= \frac{2}{15} \times C \times (2\times g)^{0.5} \times (3B_1+2B_2) \times Dh^{3/2}$$

Where, Dh : Water depth (m)

B_2 : Width of overflow (m)

In case of Dh=1.5

$$Q= \frac{2}{15} \times 0.60\times(2\times 9.81)\hat{}0.5\times(3\times 3.0+2\times 3.0)\times 1.5\hat{}(3/2)= \quad 9.76 \quad < 9.76$$

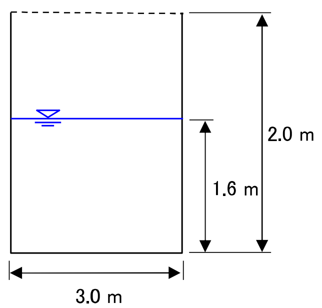
・・・・・NG

In case of Dh=1.6

$$Q= \frac{2}{15} \times 0.60\times(2\times 9.81)\hat{}0.5\times(3\times 3.0+2\times 3.0)\times 1.6\hat{}(3/2)= \quad 10.75 \quad \geqq 9.76$$

・・・・・OK

Therefore,
Dh = 1.6 m



Schematic diagram of opening section

The overflow water depth is 1.6m, and the opening height is 2m, so the target water flow can flow down only through the opening section.

DD Leprak 2

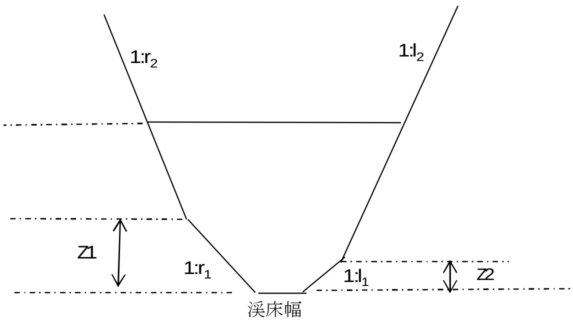
Stability calculation of S4-3 DD Leprak2
(Dam height H＜15m)

1. Design specifications

1.1 Design condition list

①Topography, geology, facility shape conditions

Symbol	Item	Unit	Value
	Topographic condition		
	Width of Riverbed	m	100.0
$1{:}r_1$	Flow section right bank slope (bottom)		1.0
$1{:}r_2$	Flow section right bank slope (top)		1.0
$1{:}l_1$	Flow section left bank slope (bottom)		1.0
$1{:}l_2$	Flow section left bank slope (top)		1.0
θ	Riverbed surface slope		1/33.3
	Flow rate conditions		
A	Target basin area	km^2	25.80
P_{24}	24-hour(day) rainfall	mm	251.0
	Facility shape conditions		
H	Dam height	m	5.0
B	Crest width	m	3.0
b1	Road width	m	0.0
b2	Wing crest width	m	3.0
m	Upstream slope(overflow section)		0.30
m	Upstream slope(non-overflow section)		0.30
n	Downstream slope		0.20
B1	Base width of spillway	m	815.0
	Wing edge		0.5
	Sediment conditions		
d_{95}	Maximum boulder size	m	2.5
	Ground conditions		
τ_0	Shear strength		0
f	Friction coefficient		0.60
qu	Allowable bearing capacity	kN/m^2	600
n	Safety factor against sliding		1.2



②Design constant

We	Apparent unit weight of concrete	kN/m^3	20
We	Unit weight of concrete	kN/m^3	22.6
Wo	Unit weight of water	kN/m^3	11.77
σ	Gravel density	kg/m^3	2,600
ρ	Water density	kg/m^3	1,200
K_{p1}	Coefficient		120
C	Coefficient of overflow		0.6
g	Gravitational acceleration	m/s^2	9.81
C_*	metric concentration of deposited sediment		0.6
Kn	Roughness coefficient(front part)		0.1
C	Coefficient of earth pressure		0.3
ϕ	Internal friction angle of sediment	degree	35.0
τ c	Shear strength of concrete		2,760

Regarding the shear strength, the smaller value in between the dambody and the ground is used in calculation.

1.2 Apparent unit weight of concrete ※1

Symbol	Item	Unit	Value
B1	Base width of spillway	m	815.0
b	Width of opening section	m	3.0
h	Height of opening section	m	2.0
h1	Slab above the opening	m	0.0
	Number of openings	parts	38
W	Dam body volume except opening	m3	4,296
Vc	Dam body volume including opening	m3	5,094
Wc	Unit weight of concrete	kN/m3	22.6
Wγc	Apparent unit weight of concrete	kN/m3	19.06

The apparent unit weight of concrete adopts 20.0 kN/m3

⇒ Adopted value : 20.0 kN/m3

3) If the permeable part is made of concrete, the self-weight of the dam body is calculated using the dam body block volume (V_c) which is calculated by assuming the overflow section as a closed structure, and the dam body block weight (W_{rc}) which is calculated by assuming the overflow section as an open structure (Figure 11). Further, it is noted that the dam body block refers to the concrete block in the permeable part, not the concrete block used at the time of casting.

$$\gamma_{rc} = W_{rc} / V_C \quad \text{.......(6)}$$

Where,

γ_{rc} : apparent unit weight of concrete (kN/m^3),

W_{rc} : dam body block weight (kN) calculated by assuming the overflow section as an open structure with concrete,

V_c : dam body block volume (m^3) calculated by assuming the overflow section as a closed structure.

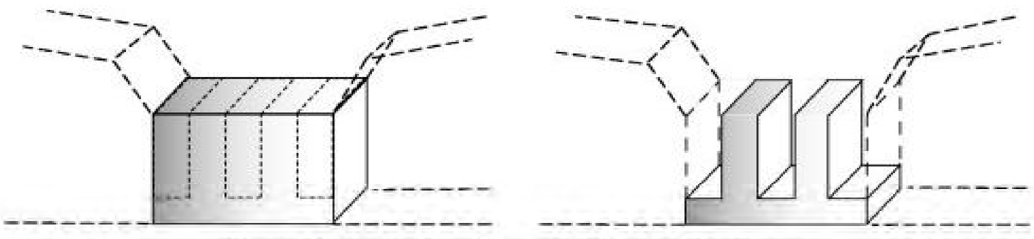


Figure 11 Dam body volume of spillway of the slit part

4) Combinations of design external forces excluding the self-weight of dam body are as shown in Table 4.

※1 Source : Technical Guideline for Designing Sabo Facilities

2. Calculation of flow discharge considering sediment content (1.5Qp)

2.1 Flow discharge considering sediment content

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Target basin area	A	km^2	25.80

The flow discharge considering sediment content is calculated by under rainfall(251mm/24h) with the design exceedance probability.
The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Qp = \frac{1}{3.6} \times K_{f1} \times Pa \times A$$ ※1

Qp : Peak flood discharge (m^3/s)
Pa : Average rainfall intensity during the flood concentration time(mm/hr)
A : Target basin area(km^2)
K_{f1} : Runoff coefficient

Therefore, the peak discharge of water without sediment is as follow.

Qp＝ 371 (m^3/s)

Flow discharge considering sediment content is 1.5 times the peak discharge of water without sediment, and calculated as follow.

Q= 1.5 × Qp ※1
= 1.5 × 371

= 556.5 (m^3/s)

※1 Source : Technical Guideline for Establishing Sabo Master Plan

2.2 Value of overflow depth in regards to the discharge bulked with sediment

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Discharge of water considering sediment content	Q	m^3/s	556.50
Coefficient of overflow	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Base width of the spillway	B1	m	815.0
Wing edge slope	m		0.5

The overflow depth in regards to discharge of water containing fine sediments is calculated as follow.

$$Q = \frac{2}{15} \times C \times (2 \times g)^{0.5} \times (3B_1 + 2B_2) \times Dh^{3/2}$$ ※1

Where,

Dh : Overflow depth (m)

B_2 : Width of overflow (m)

In case of Dh=0.5

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 815.0 + 2 \times 815.5) \times 0.5\hat{}(3/2) \quad 510.65 \quad < 556.50$$

・・・・・NG

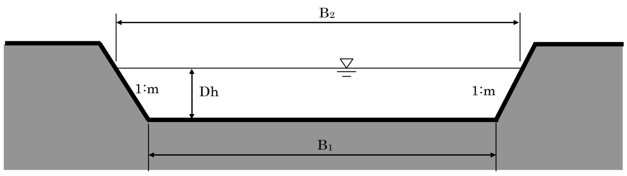
In case of Dh=0.6

$$Q = \frac{2}{15} \times 0.60 \times (2 \times 9.81)\hat{}0.5 \times (3 \times 815.0 + 2 \times 815.6) \times 0.6\hat{}(3/2) \quad 671.30 \quad \geqq 556.50$$

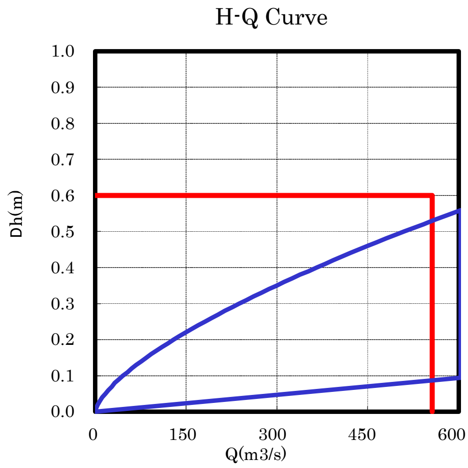
・・・・・OK

Therefore,

Dh ＝ 0.6 m



Schematic diagram of the cross-sectional shape of Spillway



※1 Source : SNI 2851-2021_Desain Saboda

3. Calculation of Peak discharge of debris flow (Qsp)

3.1 Peak discharge of debris flow (Qsp) ※1

Specifications list

Symbol	Item	Unit	Value
σ	Density of gravel	kg/m^3	2,600
ρ	Density of water	kg/m^3	1,200
Wo	Unit weight of the water	kN/m^3	11.77
ϕ	Internal friction angle of the deposited soil	degrees	35.0
d	Maximum boulder size	m	2.50
$\tan\theta$	Riverbed surface slope		1/33.3
C*	Volumetric concentration of deposited sediment		0.60
Kn	Roughness coefficient (the front part of debris flow)		0.10
B	Width of riverbed	m	100.0

3.1.1 Concentration of debris flow (Takahashi's Equation)

$$Cd = \frac{\rho \cdot \tan\theta}{(\sigma - \rho)\cdot(\tan\phi - \tan\theta)}$$

Where,

ρ	;	Density of water	1,200	(Kg/m^3)
θ	;	Riverbed surface slope	1.72	(degrees) (i=1/33.3)
σ	;	Density of gravel	2,600	(Kg/m^3)
ϕ	;	Internal friction angle of sediment deposited on riverbed	35.0	(degrees)

$$= \frac{1{,}200 \times \tan 1.72)}{(2{,}600 - 1{,}200) \times (\tan 35.0 - \tan 1.72)}$$

$$= 0.04$$

When the calculated value (Cd) is larger than 0.9C*, , Cd is assumed to be equal to 0.9C*. Meanwhile, if the calculated value (Cd) is smaller than 0.3, Cd value is assumed to be 0.3

$\therefore$ Cd = 0.30

3.1.2 Peak discharge of debris flow (Qsp)

The peak discharge of debris flow Qsp (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Qp (m^3/s)

$$Qsp = \frac{C*}{C*-Cd} \times Qp$$

Where,

Cd	:	Concentration of debris flow	=	0.30	
C*	:	Volumetric concentration of deposited sediment	=	0.60	
Qp	:	Peak discharge of water	=	371.00	(m^3/s)

$$Qsp = \frac{C*}{C*-Cd} \times Qp = \frac{0.60}{0.60 - 0.30} \times 371 = 742.0$$

Qsp= 742.0 (m^3/sec)

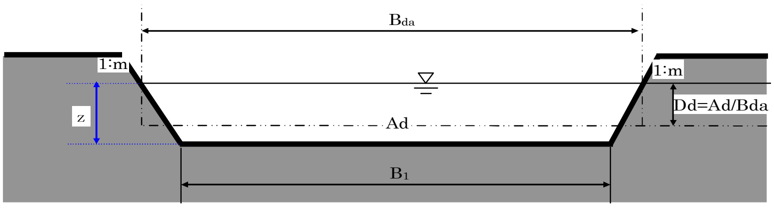
※1 Source : SNI 2851-2021_Desain Sabodam

3.2 Value of the overflow depth for debris flow peak discharge (Qsp).

3.2.1 Calculation of overflow depth ※1

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Debris flow peak discharge	Qsp	m^3/s	742.0
Roughness coefficient	Kn		0.1
Design deposit surface slope	θ	degrees	1.43
Base width of spillway	B1	m	815.0
Wing edge slope	m		0.5



The amount of the debris flow that can flow down through this cross section is determined as follow.

Qsp=U・Ad

Where,

z :	Debris flow surface water level	(m)
U :	Debris flow velocity	(m/s)
Bda :	Width of the flow	(m)
Ad :	Cross-sectional aera of debris flow peak discharge	(m^2)
Dd :	Debris flow depth	(m)

U, Bda, Ad and Dd can be calculated as follow.

$U= 1/Kn \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2}$

$Dd= Ad/Bda$

$Ad= (B1+m \times z) \times z$

$Bda= B1+2 \times m \times z$

If the debris flow depth is 0.71m

Qsp= 1.26 × 578.90

= 729.4 < 742.0 ・・・・・NG

$U= 1/0.1 \times 0.71^{2/3} \times 0.02^{1/2} = 1.26$

$Dd= 578.90 / 815.71 = 0.71$

$Ad= (815.0 +0.5 \times 0.71) \times 0.71 = 578.90$

$Bda= 815.0 +2 \times 0.50 \times 0.71 = 815.71$

If the debris flow depth is 0.72m

Qsp= 1.27 × 587.06

= 745.5 ≧ 742.0 ・・・・・OK

$U= 1/0.1 \times 0.72^{2/3} \times 0.02^{1/2} = 1.27$

$Dd= 587.06 / 815.72 = 0.72$

$Ad= (815.0 +0.5 \times 0.72) \times 0.72 = 587.06$

$Bda= 815.0 +2 \times 0.50 \times 0.72 = 815.72$

Therefore, the debris flow depth is adapted as 0.8m

※1 Source : Technical Guideline for Establishing Sabo Master Plan

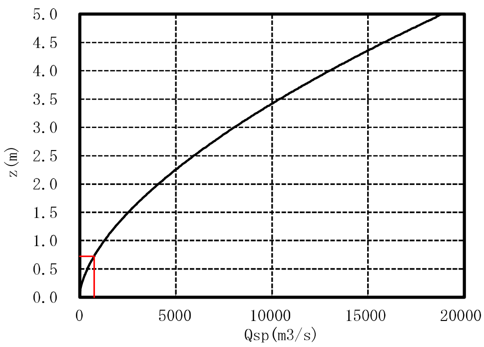
3.2.2 Debris flow specifications over spillway

$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

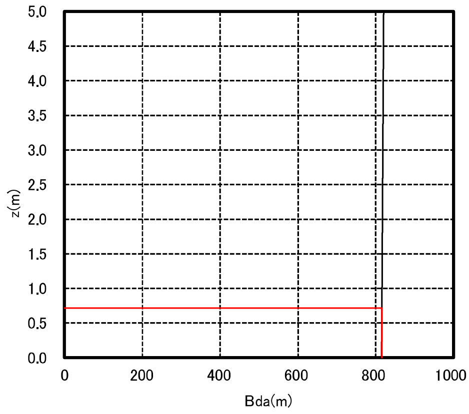
Debris flow discharge (Qsp) 742.0 m^3/s
Design deposit surface slope 1/40.0

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
0.47	815.0	815.46	375.01	0.46	0.1	1.43	0.9	352.5
0.48	815.0	815.47	383.16	0.47	0.1	1.43	1.0	364.0
0.49	815.0	815.48	391.32	0.48	0.1	1.43	1.0	379.5
0.50	815.0	815.49	399.47	0.49	0.1	1.43	1.0	391.4
0.51	815.0	815.50	407.63	0.50	0.1	1.43	1.0	407.6
0.52	815.0	815.51	415.78	0.51	0.1	1.43	1.0	419.9
0.53	815.0	815.52	423.94	0.52	0.1	1.43	1.0	432.4
0.54	815.0	815.53	432.09	0.53	0.1	1.43	1.0	445.0
0.55	815.0	815.54	440.25	0.54	0.1	1.43	1.1	462.2
0.56	815.0	815.55	448.40	0.55	0.1	1.43	1.1	475.3
0.57	815.0	815.56	456.56	0.56	0.1	1.43	1.1	488.5
0.58	815.0	815.57	464.71	0.57	0.1	1.43	1.1	506.5
0.59	815.0	815.58	472.87	0.58	0.1	1.43	1.1	520.1
0.60	815.0	815.59	481.02	0.59	0.1	1.43	1.1	533.9
0.61	815.0	815.60	489.18	0.60	0.1	1.43	1.1	547.8
0.62	815.0	815.61	497.34	0.61	0.1	1.43	1.1	566.9
0.63	815.0	815.62	505.49	0.62	0.1	1.43	1.2	581.3
0.64	815.0	815.63	513.65	0.63	0.1	1.43	1.2	595.8
0.65	815.0	815.64	521.80	0.64	0.1	1.43	1.2	610.5
0.66	815.0	815.65	529.96	0.65	0.1	1.43	1.2	630.6
0.67	815.0	815.66	538.12	0.66	0.1	1.43	1.2	645.7
0.68	815.0	815.67	546.27	0.67	0.1	1.43	1.2	660.9
0.69	815.0	815.69	562.59	0.69	0.1	1.43	1.2	691.9
0.70	815.0	815.70	570.75	0.70	0.1	1.43	1.3	713.4
0.71	815.0	815.71	578.90	0.71	0.1	1.43	1.3	729.4
0.72	815.0	815.72	587.06	0.72	0.1	1.43	1.3	745.5
0.73	815.0	815.73	595.22	0.73	0.1	1.43	1.3	761.8
0.74	815.0	815.74	603.37	0.74	0.1	1.43	1.3	778.3
0.75	815.0	815.75	611.53	0.75	0.1	1.43	1.3	794.9
0.76	815.0	815.76	619.69	0.76	0.1	1.43	1.3	817.9
0.77	815.0	815.77	627.85	0.77	0.1	1.43	1.3	835.0
0.78	815.0	815.78	636.00	0.78	0.1	1.43	1.3	852.2
0.79	815.0	815.79	644.16	0.79	0.1	1.43	1.4	869.6
0.80	815.0	815.80	652.32	0.80	0.1	1.43	1.4	887.1
0.81	815.0	815.81	660.48	0.81	0.1	1.43	1.4	904.8
0.82	815.0	815.82	668.64	0.82	0.1	1.43	1.4	922.7
0.83	815.0	815.83	676.79	0.83	0.1	1.43	1.4	947.5
0.84	815.0	815.84	684.95	0.84	0.1	1.43	1.4	965.7
0.85	815.0	815.85	693.11	0.85	0.1	1.43	1.4	984.2
0.86	815.0	815.86	701.27	0.86	0.1	1.43	1.4	1002.8
0.87	815.0	815.87	709.43	0.87	0.1	1.43	1.4	1021.5
0.88	815.0	815.88	717.59	0.88	0.1	1.43	1.5	1040.5
0.89	815.0	815.89	725.75	0.89	0.1	1.43	1.5	1059.5
0.90	815.0	815.90	733.91	0.90	0.1	1.43	1.5	1078.8
0.91	815.0	815.91	742.06	0.91	0.1	1.43	1.5	1098.2
0.92	815.0	815.92	750.22	0.92	0.1	1.43	1.5	1117.8
0.93	815.0	815.93	758.38	0.93	0.1	1.43	1.5	1145.1
0.94	815.0	815.94	766.54	0.94	0.1	1.43	1.5	1165.1
0.95	815.0	815.95	774.70	0.95	0.1	1.43	1.5	1185.2
0.96	815.0	815.96	782.86	0.96	0.1	1.43	1.5	1205.6
0.97	815.0	815.97	791.02	0.97	0.1	1.43	1.6	1226.0

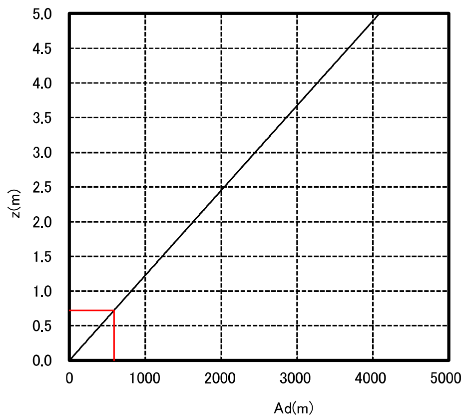
0.72	815.0	815.72	587.06	0.72	0.1	1.43	1.3	745.5



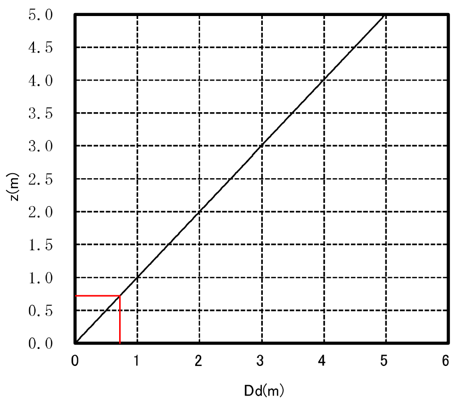
Q_{sp}-Z Curve

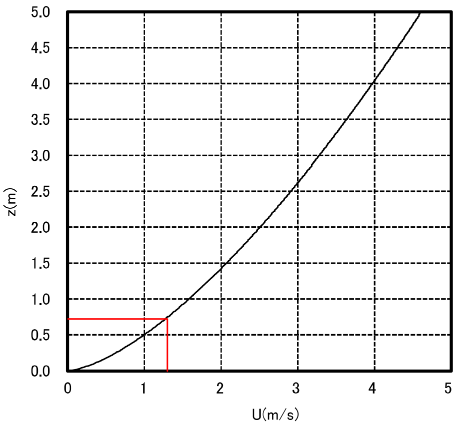


B_{da}-z Curve



A_d-Z Curve





Dd-Z Curve

U-Z Curve

4. Design water depth※1

The design water depth is the largest of the values from the calculation results of below chart.
Therefore, since the Maximum boulder size has the largest value, the design water depth is determined to be 2.5m.

Table 1 Overflow water depth calculated by each method

Item	Depth(m)
Value of the overflow depth of debris flow peak discharge	0.8
Value of overflow depth of discharge considering sediment content	0.6
Maximum boulder size	2.5

The freeboard is set based on Table 2. However, the freeboard height shall be designed according to the riverbed slope and freeboard height ratio to the design water depth must not be less than the value presented in Table 3. The riverbed slope referred here is the design sediment deposition slope.

Table 2 Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6 m
200 ～ 500 m^3/s	0.8 m
500 m^3/s or more	1.0 m

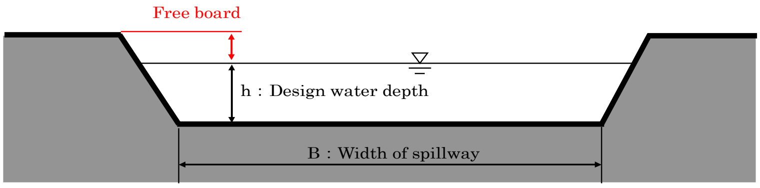


Table 3 Minimum value of the ratio of the freeboard height to the design water depth by riverbed slope

Riverbed slope	(Freeboard height)/(Design water depth)
1/10 or more	0.50
1/10～1/30	0.40
1/30～1/50	0.30
1/50～1/70	0.25

Design deposit surface slope : 1/40
Lowest value of (Freeboard height)/(Design water depth) : 0.3

Since the design discharge is 556.5m3/s, the freeboard is 1.0 m
(Freeboard height)/(Design water depth) is 1m / 2.5m = 0.40 ＞ 0.3
(Minimum required freeboard height = 0.75m)

Therefore, the freeboard height adopts 1.0m

As a result of the above, Spillway height is 2.5 + 1 = 3.5 m

However, the spillway height will be adjusted to match the height of the existing Dike, and a height of 8.0 m will be adopted.

※1 Source : SNI 2851-2021_Desain Sabodam

5. Specifications of debris flow

5.1 Setting of debris flow velocity and depth based on debris flow peak discharge

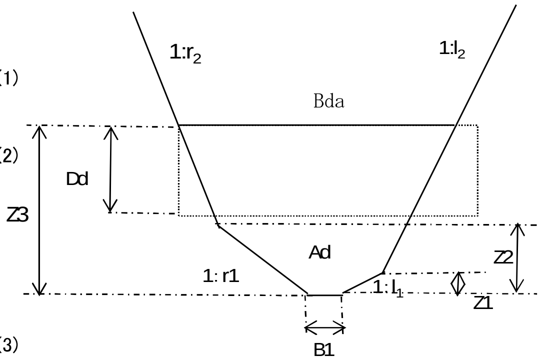
5.1.1 Debris flow cross–section

n_{r1}	:	Flow section right bank slope (bottom)	1.00
n_{r2}	:	Flow section right bank slope (top)	1.00
n_{l1}	:	Flow section left bank slope (bottom)	1.00
n_{l2}	:	Flow section left bank slope (top)	1.00
z	:	Debris flow surface water level	
Bda	:	Width of the flow	

Bda= $n_{r1} \times z + n_{l1} \times z$ (0<z<z1)

$n_{r1} \times z + n_{l2} \times z$ (z1<z<z2)

$n_{r2} \times z + n_{l2} \times z$ (z2<z) ･･･(1)



5.1.2 debris flow depth (Dd)

Dd= Ad ／ Bda ･･･(2)

Ad : Cross–sectional aera of debris flow peal

Ad= Ad1 (0<z<z1)

Ad2 (z1<z<z2)

Ad3 (z2<z)

5.1.3 Debris flow velocity (U)

$$U = \frac{1}{Kn} \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2} \quad \cdots(3)$$

Kn	:	Roughness coefficient	0.10		
θ	:	Riverbed surface slope	1.72	(degrees)	(i=1／33.3)

5.1.4 Calculate debris flow velocity and depth

$Q_{special}$ = U・Ad ･･･(4)

From formulas (1), (2), (3), and (4), when z is calculated when Qspecial matches Qsp, z is as follows.

z= 2.39m

Calculating debris flow velocity (U) and depth (Dd) from the result of z above, U and Dd are as follows.

Dd= 2.34m , U= 3.05m/s

5.2 Unit weight of debris flow (γd)

γd= (σ × Cd)+{ρ × (1 − Cd)}

= {(2,600 × 0.30)+(1,200 × (1− 0.30)} × 9.81/1000

= 15.89 (kN/m^3)

5.3 Drag force of debris flow (F)

$$F = \alpha \cdot \frac{\gamma_d}{g} \cdot Dd \cdot U^2$$

$$= 1.0 \times \frac{15.89}{9.81} \times 2.34 \times 3.05^2$$

= 35.26 (KN／m)

5.4 Unit weight of debris flow in water (ρdi)

ρdi=γd−Wo

= 15.89 − 11.77 = 4.12 (kN/m3)

5.5 Unit weight of sediment in water (γs)

γs=C*(σ−ρ)× g/1000

= 0.6 ×(2,600 − 1,200)× 9.81/1000 = 8.24 (kN/m3)

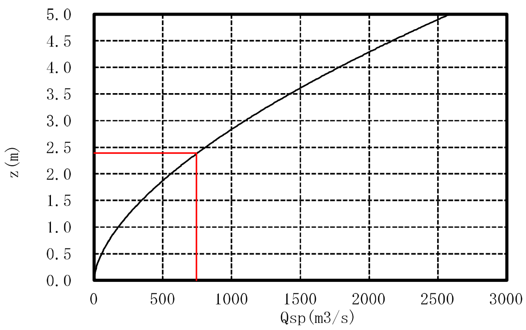
5.6 Debris flow specifications over Riverbed

Debris flow discharge (Qsp) 742.0 m^3/s
Riverbed surface slope 1/33.3

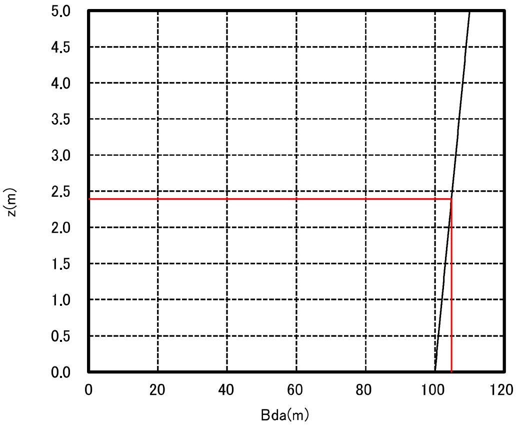
$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
2.14	100.0	104.30	218.58	2.10	0.1	1.72	2.84	620.7
2.15	100.0	104.30	219.62	2.11	0.1	1.72	2.85	625.9
2.16	100.0	104.30	220.67	2.12	0.1	1.72	2.86	631.1
2.17	100.0	104.30	221.71	2.12	0.1	1.72	2.86	634.0
2.18	100.0	104.40	222.75	2.13	0.1	1.72	2.87	639.2
2.19	100.0	104.40	223.80	2.14	0.1	1.72	2.88	644.5
2.20	100.0	104.40	224.84	2.15	0.1	1.72	2.89	649.7
2.21	100.0	104.40	225.88	2.16	0.1	1.72	2.89	652.7
2.22	100.0	104.40	226.93	2.17	0.1	1.72	2.90	658.0
2.23	100.0	104.50	227.97	2.18	0.1	1.72	2.91	663.3
2.24	100.0	104.50	229.02	2.19	0.1	1.72	2.92	668.7
2.25	100.0	104.50	230.06	2.20	0.1	1.72	2.93	674.0
2.26	100.0	104.50	231.11	2.21	0.1	1.72	2.94	679.4
2.27	100.0	104.50	232.15	2.22	0.1	1.72	2.95	684.8
2.28	100.0	104.60	233.20	2.23	0.1	1.72	2.96	690.2
2.29	100.0	104.60	234.24	2.24	0.1	1.72	2.97	695.6
2.30	100.0	104.60	235.29	2.25	0.1	1.72	2.97	698.8
2.31	100.0	104.60	236.34	2.26	0.1	1.72	2.98	704.2
2.32	100.0	104.60	237.38	2.27	0.1	1.72	2.99	709.7
2.33	100.0	104.70	238.43	2.28	0.1	1.72	3.00	715.2
2.34	100.0	104.70	239.48	2.29	0.1	1.72	3.01	720.8
2.35	100.0	104.70	240.52	2.30	0.1	1.72	3.02	726.3
2.36	100.0	104.70	241.57	2.31	0.1	1.72	3.03	731.9
2.37	100.0	104.70	242.62	2.32	0.1	1.72	3.04	737.5
2.38	100.0	104.80	243.66	2.33	0.1	1.72	3.04	740.7
2.39	100.0	104.80	244.71	2.34	0.1	1.72	3.05	746.3
2.40	100.0	104.80	245.76	2.35	0.1	1.72	3.06	752.0
2.41	100.0	104.80	246.81	2.35	0.1	1.72	3.06	755.2
2.42	100.0	104.80	247.86	2.36	0.1	1.72	3.07	760.9
2.43	100.0	104.90	248.90	2.37	0.1	1.72	3.08	766.6
2.44	100.0	104.90	249.95	2.38	0.1	1.72	3.09	772.3
2.45	100.0	104.90	251.00	2.39	0.1	1.72	3.10	778.1
2.46	100.0	104.90	252.05	2.40	0.1	1.72	3.11	783.8
2.47	100.0	104.90	253.10	2.41	0.1	1.72	3.11	787.1
2.48	100.0	105.00	254.15	2.42	0.1	1.72	3.12	792.9
2.49	100.0	105.00	255.20	2.43	0.1	1.72	3.13	798.7
2.50	100.0	105.00	256.25	2.44	0.1	1.72	3.14	804.6
2.51	100.0	105.00	257.30	2.45	0.1	1.72	3.15	810.4
2.52	100.0	105.00	258.35	2.46	0.1	1.72	3.16	816.3
2.53	100.0	105.10	259.40	2.47	0.1	1.72	3.17	822.2
2.54	100.0	105.10	260.45	2.48	0.1	1.72	3.17	825.6
2.55	100.0	105.10	261.50	2.49	0.1	1.72	3.18	831.5
2.56	100.0	105.10	262.55	2.50	0.1	1.72	3.19	837.5
2.57	100.0	105.10	263.60	2.51	0.1	1.72	3.20	843.5
2.58	100.0	105.20	264.66	2.52	0.1	1.72	3.21	849.5
2.59	100.0	105.20	265.71	2.53	0.1	1.72	3.22	855.5
2.60	100.0	105.20	266.76	2.54	0.1	1.72	3.23	861.6
2.61	100.0	105.20	267.81	2.55	0.1	1.72	3.23	865.0
2.62	100.0	105.20	268.86	2.55	0.1	1.72	3.23	868.4
2.63	100.0	105.30	269.92	2.56	0.1	1.72	3.24	874.5
2.64	100.0	105.30	270.97	2.57	0.1	1.72	3.25	880.6

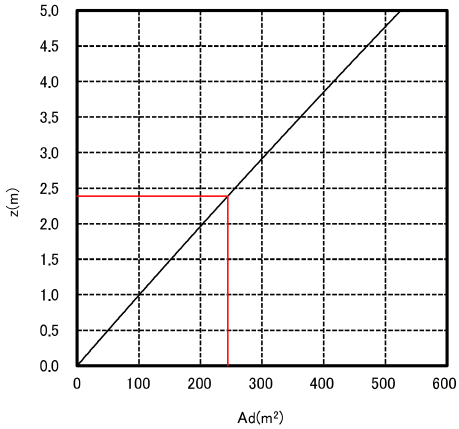
2.39	100.0	104.78	244.71	2.34	0.1	1.72	3.05	746.3



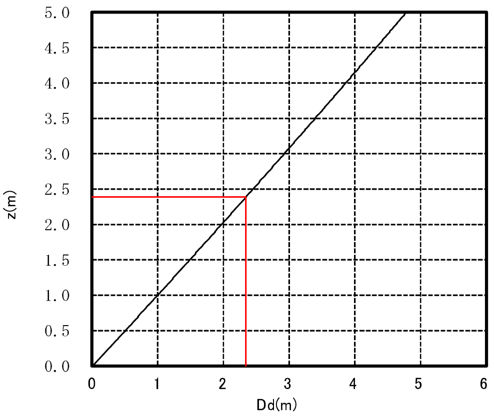
Q_{sp}-Z Curve



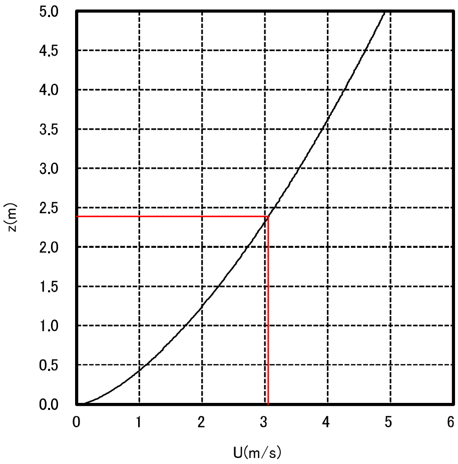
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

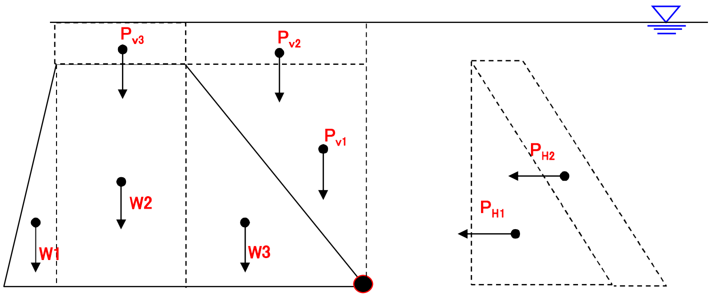
6. Stability analysis

6.1 Result of Stability analysis (overflow section)

During flooding					
Stability against overturning	e	0.824	≦	0.917	OK
Stability against sliding	n	1.22	≧	1.2	OK
Maximum ground reaction force	σ max	207.7	≦	600	OK
Minimum ground reaction force	σ min	11.1	≧	0	OK
During debris flow					
Stability against overturning	e	0.390	≦	0.917	OK
Stability against sliding	n	1.76	≧	1.2	OK
Maximum ground reaction force	σ max	126.8	≦	600	OK
Minimum ground reaction force	σ min	51.1	≧	0	OK

6.2 Verification of stability of main body (overflow section)

6.2.1 During flooding



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 5.50 − 2/3 × 1.00	50.000		4.833	241.667
W2	3.00 × 5.00 × 20.00 1.50 + 1/2 × 3.00	300.000		3.000	900.000
W3	1/2 × 1.50 × 5.00 × 20.00 2/3 × 1.50	75.000		1.000	75.000
Hydrostatic pressure P_{V1}	1/2 × 1.50 × 5.00 × 11.77 1/3 × 1.50	44.138		0.500	22.069
P_{V2}	1.50 × 2.50 × 11.77 1/2 × 1.50	44.138		0.750	33.103
P_{V3}	3.00 × 2.50 × 11.77 1.50 + 1/2 × 3.00	88.275		3.000	264.825
P_{H1}	1/2 × 5.00 × 5.00 × 11.77 1/3 × 5.00		147.125	1.667	245.208
P_{H2}	2.50 × 5.00 × 11.77 1/2 × 5.00		147.125	2.500	367.813
TOTAL		601.550	294.250		2,149.685

Stability against overturning

Resultant force position $d = \dfrac{\Sigma M}{\Sigma V} = \dfrac{2149.685}{601.550} = 3.574$ m

Eccentric distance
$e = d - B/2 = 3.574 - 5.500 / 2 = 0.824$ m
$e = 0.824$ m $\leqq B/6 = 0.917$ m

OK

Stability against sliding

$$n = \frac{f \times \Sigma V + \tau \times L}{H}$$

$$= \frac{0.600 \times 601.550 + 0.0 \times 5.500}{294.250} = 1.22$$

$n = 1.22 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min = \frac{\Sigma V}{B} \times \left(1 - \frac{6 \times e}{B}\right)$$

$$= \frac{601.55}{5.50} \times \left(1 - \frac{6 \times 0.82}{5.50}\right) = 11.1 \text{ kN/m}^2 \geqq 0 \text{ kN/m}^2$$

OK

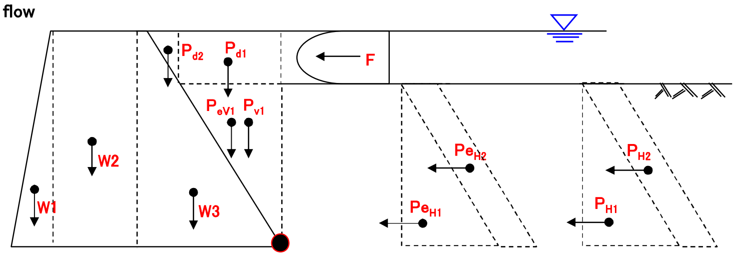
Maximum ground reaction force

$$\sigma max = \frac{\Sigma V}{B} \times \left(1 + \frac{6 \times e}{B}\right)$$

$$= \frac{601.55}{5.50} \times \left(1 + \frac{6 \times 0.82}{5.50}\right) = 207.7 \text{ kN/m}^2 \leqq 600 \text{ kN/m}^2$$

OK

6.2.2 During bebris flow



Schematic diagram of design external forces

Remarks：Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN･m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 5.50 − 2/3 × 1.00	50.000		4.833	241.667
W2	3.00 × 5.00 × 20.00 1.50 ＋ 1/2 × 3.00	300.000		3.000	900.000
W3	1/2 × 1.50 × 5.00 × 20.00 2/3 × 1.50	75.000		1.000	75.000
Hydrostatic pressure P_{V1}	1/2 × 0.80 × 2.66 × 11.77 1/3 × 0.80	12.492		0.266	3.323
P_{H1}	1/2 × 2.66 × 2.66 × 11.77 1/3 × 2.66		41.640	0.887	36.921
P_{H2}	2.34 × 2.66 × 11.77 1/2 × 2.66		73.261	1.330	97.437
Earth pressure P_{ev1}	1/2 × 0.80 × 2.66 × 8.24 1/3 × 0.80	8.745		0.266	2.326
P_{eH1}	1/2 × 0.30 × 2.66^2 × 8.24 1/3 × 2.66		8.745	0.887	7.754
P_{eH2}	0.30 × 2.34 × 2.66 × 4.12 1/2 × 2.66		7.690	1.330	10.228
Mass of debris flow Pd1	0.80 × 2.34 × 15.89 1/2 × 0.80	29.672		0.399	11.839
Pd2	1/2 × 0.70 × 2.34 × 15.89 1.50 − 2/3 × 0.70	13.051		1.032	13.469
Drag force of debris flow F	1.00 × 15.89 × 3.05^2 × 2.34 /9.81 2.66 ＋ 1/2 × 2.34		35.259	3.830	135.042
TOTAL		488.960	166.596		1,535.006

Stability against overturning

Resultant force position $d=\dfrac{\Sigma M}{\Sigma V}=\dfrac{1535.006}{488.960}=3.139$ m

Eccentric distance

$e=d-B/2 = 3.139 - 5.500 / 2 = 0.390$ m

$e= 0.390$ m $\leqq B/6= 0.917$ m ……… OK

Stability against sliding

$$n=\frac{f\times\Sigma V+\tau\times L}{H}$$

$$=\frac{0.600\times 488.960 + 0.0 \times 5.50}{166.596}=1.76$$

$n= 1.76 \geqq 1.2$ ……… OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma min=\frac{\Sigma V}{B}\times\left(1-\frac{6\times e}{B}\right)$$

$$=\frac{488.96}{5.50}\times\left(1-\frac{6\times 0.39}{5.50}\right)=51.1\ \text{kN/m}^2 \geqq 0\ \text{kN/m}^2$$

……… OK

Maximum ground reaction force

$$\sigma max=\frac{\Sigma V}{B}\times\left(1+\frac{6\times e}{B}\right)$$

$$=\frac{488.96}{5.50}\times\left(1+\frac{6\times 0.39}{5.50}\right)=126.8\ \text{kN/m}^2 \leqq 600\ \text{kN/m}^2$$

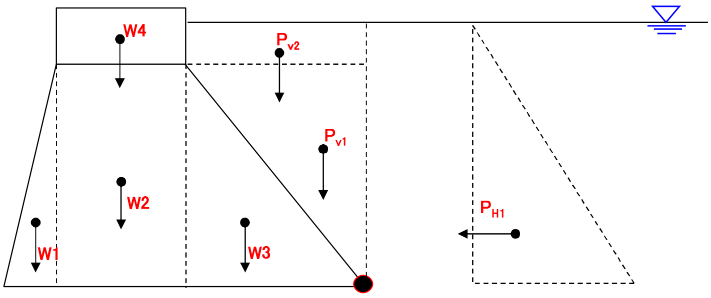
……… OK

6.3 Result of Stability analysis (non-overflow section)

During flooding					
Stability against overturning	e	0.814	≦	0.917	OK
Stability against sliding	n	1.80	≧	1.2	OK
Maximum ground reaction force	σ max	341.0	≦	600	OK
Minimum ground reaction force	σ min	20.3	≧	0	OK
During debris flow					
Stability against overturning	e	0.756	≦	0.917	OK
Stability against sliding	n	1.70	≧	1.2	OK
Maximum ground reaction force	σ max	343.7	≦	600	OK
Minimum ground reaction force	σ min	33.1	≧	0	OK

6. 4 Verification of stability of main body (non-overflow section)

6.4.1 During flooding



Schematic diagram of design external forces

Remarks : Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 5.50 − 2/3 × 1.00	50.000		4.833	241.667
W2	3.00 × 5.00 × 20.00 1.50 ＋ 1/2 × 3.00	300.000		3.000	900.000
W3	1/2 × 1.50 × 5.00 × 20.00 2/3 × 1.50	75.000		1.000	75.000
W4	8.00 × 3.00 × 20.00 1.50 ＋ 3.00 × 1/2	480.000		3.000	1,440.000
Hydrostatic pressure P_{V1}	1/2 × 1.50 × 5.00 × 11.77 1/3 × 1.50	44.138		0.500	22.069
P_{V2}	1.50 × 2.50 × 11.77 1/2 × 1.50	44.138		0.750	33.103
P_{H1}	1/2 × 7.50 × 7.50 × 11.77 1/3 × 7.50		331.031	2.500	827.578
TOTAL		993.275	331.031		3,539.417

Stability against overturning

Resultant force position $d=\dfrac{\Sigma M}{\Sigma V}=\dfrac{3539.417}{993.275}=3.563$ m

Eccentric distance
$e= d-B/2 = 3.563 - 5.500 / 2 = 0.814$ m
$e= 0.814$ m $\leqq$ $B/6= 0.917$ m

OK

Stability against sliding

$$n=\frac{f\times\Sigma V+\tau\times L}{H}$$

$$=\frac{0.600 \times 993.275 + 0.0 \times 5.500}{331.031} = 1.80$$

$n= 1.80 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma\,\mathrm{min}=\frac{\Sigma V}{B}\times\left(1-\frac{6\times e}{B}\right)$$

$$=\frac{993.28}{5.50}\times\left(1-\frac{6\times 0.81}{5.50}\right)=20.3\ \mathrm{kN/m^2}\geqq 0\ \mathrm{kN/m^2}$$

OK

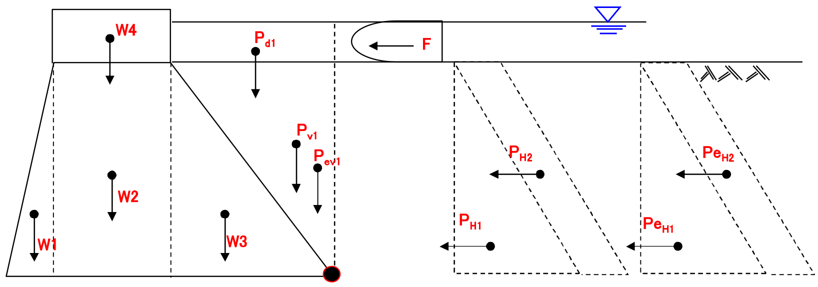
Maximum ground reaction force

$$\sigma\,\mathrm{max}=\frac{\Sigma V}{B}\times\left(1+\frac{6\times e}{B}\right)$$

$$=\frac{993.28}{5.50}\times\left(1+\frac{6\times 0.81}{5.50}\right)=341.0\ \mathrm{kN/m^2}\leqq 600\ \mathrm{kN/m^2}$$

OK

6.4.2 During bebris flow



Schematic diagram of design external forces

Remarks: Upper part of the calculation formula is the calculation of force
Lower part of the calculation formula is the calculation of arm length

Classification	Calculation formula	Vertical force (kN)	horizontal force (kN)	Arm length (m)	Moment (kN・m)
Self-weight of dam W1	1/2 × 1.00 × 5.00 × 20.00 5.50 − 2/3 × 1.00	50.000		4.833	241.667
W2	3.00 × 5.00 × 20.00 1.50 + 1/2 × 3.00	300.000		3.000	900.000
W3	1/2 × 1.50 × 5.00 × 20.00 2/3 × 1.50	75.000		1.000	75.000
W4	8.00 × 3.00 × 20.00 1.50 + 3.00 × 1/2	480.000		3.000	1,440.000
Hydrostatic pressure P_{V1}	1/2 × 1.50 × 5.00 × 11.77 1/3 × 1.50	44.138		0.500	22.069
P_{H1}	1/2 × 5.00 × 5.00 × 11.77 1/3 × 5.00		147.125	1.667	245.208
P_{H2}	2.34 × 5.00 × 11.77 1/2 × 5.00		137.709	2.500	344.273
Earth pressure P_{ev1}	1/2 × 1.50 × 5.00 × 8.24 1/3 × 1.50	30.900		0.500	15.450
P_{eH1}	1/2 × 0.30 × 5.00^2 × 8.24 1/3 × 5.00		30.900	1.667	51.500
P_{eH2}	0.30 × 2.34 × 5.00 × 4.12 1/2 × 5.00		14.460	2.500	36.150
Mass of debris flow Pd1	1.50 × 2.34 × 15.89 1/2 × 1.50	55.774		0.750	41.831
Drag force of debris flow F	1.00 × 15.89 × 3.05^2 × 2.34 /9.81 5.00 + 1/2 × 2.34		35.259	6.170	217.548
TOTAL		1,035.812	365.453		3,630.696

Stability against overturning

Resultant force position $d=\dfrac{\Sigma M}{\Sigma V} = \dfrac{3630.696}{1035.812} = 3.505$ m

Eccentric distance

$e = d - B/2 = 3.505 - 5.500 / 2 = 0.756$ m

$e = 0.756$ m $\leqq B/6 = 0.917$ m OK

Stability against sliding

$$n=\frac{f\times\Sigma V+\tau\times L}{H}$$

$$=\frac{0.600 \times 1035.812 + 0.0 \times 5.500}{365.453} = 1.70$$

$n = 1.70 \geqq 1.2$

OK

Stability of bearing capacity

Minimum ground reaction force

$$\sigma\,min=\frac{\Sigma V}{B}\times\left(1-\frac{6\times e}{B}\right)$$

$$=\frac{1035.81}{5.50}\times\left(1-\frac{6\times 0.76}{5.50}\right) = 33.1\ \mathrm{kN/m^2} \geqq 0\ \mathrm{kN/m^2}$$

OK

Maximum ground reaction force

$$\sigma\,max=\frac{\Sigma V}{B}\times\left(1+\frac{6\times e}{B}\right)$$

$$=\frac{1035.81}{5.50}\times\left(1+\frac{6\times 0.76}{5.50}\right) = 343.7\ \mathrm{kN/m^2} \leqq 600\ \mathrm{kN/m^2}$$

OK

7. Downstream riverbed protection works[※1]

Length of the apron（L）is calculated as follow.

$L=(1.5\sim2.0)\times(H_1+H_3)$

L=1.5×(H1＋H3)＝	6.8	m
L=2.0×(H1＋H3)＝	9.0	m

Where, H_1 : Height from the top of the apron to the base of the spillway (m)

h_3 : Design water depth(m)

⇒ Adopted value : 20.0 m

※1 Source : SNI 2851-2021_Desain Sabodam

8. Ability to flow down of the opening section

Specifications list

Item	Specifications		
	Symbol	Unit	Value
Number of opening section		Parts	38
Peak discharge of water	Q	m^3/s	371
Target discharge amount per part		$m^3/s/part$	9.76
Coefficient	C		0.60
Gravitational acceleration	g	m/s^2	9.81
Width of opening section	B1	m	3.0

The peak discharge of water without sediment is obtained by Rational Equation as follow.

$$Q=\frac{2}{15}\times C\times(2\times g)^{0.5}\times(3B_1+2B_2)\times Dh^{3/2}$$

Where, Dh : Water depth (m)

B_2 : Width of overflow (m)

In case of Dh=1.5

$$Q=\frac{2}{15}\times 0.60\times(2\times 9.81)\hat{}0.5\times(3\times 3.0+2\times 3.0)\times 1.5\hat{}(3/2)= \quad 9.76 \quad < 9.76$$

・・・・・NG

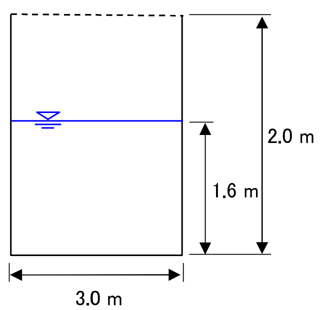
In case of Dh=1.6

$$Q=\frac{2}{15}\times 0.60\times(2\times 9.81)\hat{}0.5\times(3\times 3.0+2\times 3.0)\times 1.6\hat{}(3/2)= \quad 10.75 \quad \geqq 9.76$$

・・・・・OK

Therefore,

Dh ＝ 1.6 m



Schematic diagram of opening section

The overflow water depth is 1.6m, and the opening height is 2m, so the target water flow can flow down only through the opening section.

Appendix 4-3-2 Result of Preliminary Detailed Design(Draft) (Training Dike)

S4 Area
Tanggul　Leprak 24、25, 26

<u>Training dike</u>

S4 Area
Tanggul Leprak 24、25、26

1. Design condition list

①Topographic conditions

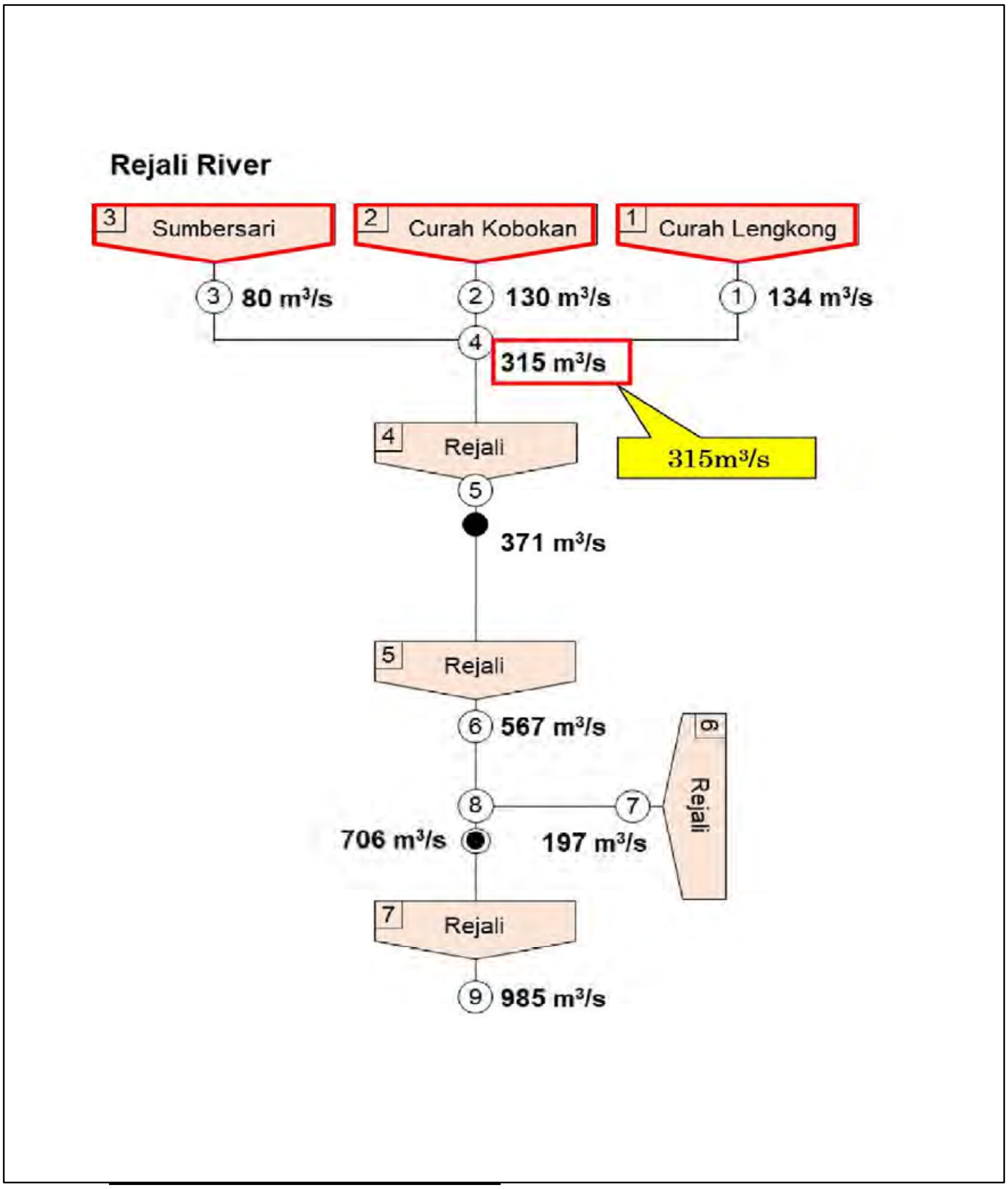
Symbol	Item	Unit	Value
	Topographic condition		
θ	Riverbed surface slope		1/20
	Flow rate conditions		
A	Target basin area	km^2	19.60

②Design constant

Wo	Unit weight of water	kN/m^3	11.77
σ	Gravel density	kg/m^3	2,600
ρ	Water density	kg/m^3	1,200
K_{p1}	Coefficient		120
C	Coefficient of overflow		0.6
g	Gravitational acceleration	m/s^2	9.81
C_*	Volumetric concentration of deposited sediment		0.6
Kn	Roughness coefficient(front part)		0.067
ϕ	Internal friction angle of sediment	degree	35.0

2. Calculation of design discharge

2.1 Peak discharge of water(Qp)



Therefore, the peak discharge of water without sediment is as follows.

= 315 (m^3/s)

2.2 Debris flow peak discharge (Qsp)

Specifications list

Symbol	Item	Unit	Value
σ	Density of gravel	kg/m^3	2,600
ρ	Density of water	kg/m^3	1,200
Wo	Unit weight of the water	kN/m^3	11.77
ϕ	Internal friction angle of the deposited soil	degrees	35.0
$\tan\theta$	Riverbed surface slope		1/20
C_*	Volumetric concentration of deposited sediment		0.60
Kn	Roughness coefficient (the front part of debris flow)		0.07

2.2.1 Concentration of debris flow (Takahashi's Equation)

$$Cd = \frac{\rho \cdot \tan\theta}{(\sigma - \rho)\cdot(\tan\phi - \tan\theta)} \quad ※1$$

Where,

ρ	;	Density of water	1,200.00	(Kg/m^3)
θ	;	Riverbed surface slope	2.86	(degrees) (i=1/20)
σ	;	Density of gravel	2,600	(Kg/m^3)
ϕ	;	Internal friction angle of sediment deposited on riverbed	35.0	(degrees)

$$= \frac{1{,}200 \times \tan 2.86}{(2{,}600 - 1{,}200) \times (\tan 35.0 - \tan 2.86)}$$

$= 0.07$

When the calculated value (Cd) is larger than 0.9C $_*$, Cd is assumed to be equal to 0.9C$_*$. Meanwhile, if the calculated value (Cd) is smaller than 0.3, Cd value is assumed to be 0.3

$\therefore$ Cd = 0.30

※1 Source : SNI 2851-2021_Desain Sabodam

2.2.2 Calculate debris flow peak discharge (Qsp)

The peak discharge of debris flow Qsp (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Qp (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp \quad ※1$$

Where,

Cd	:	Concentration of debris flow	=	0.30
C*	:	Volumetric concentration of sediment deposited on riverbed	=	0.60
Qp	:	Peak discharge of water	=	315 (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp = \frac{0.60}{0.60 - 0.30} \times 315 = 630.0$$

Qsp= 630.0 (m^3/sec)

※1 Source : SNI 2851-2021_Desain Sabodam

3. Specifications of debris flow

3.1 Setting of debris flow velocity and depth based on debris flow peak discharge ※1

3.1.1 Debris flow cross-section

The riverbed width (B1) was calculated using the following formula.
For the coefficient α, the median value (α: 4) was adopted.

$B1 = \alpha \times Q^{0.5}$

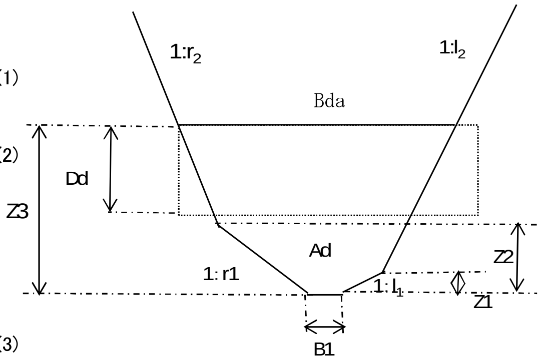
$= 4 \times 630^{0.5} = 100$ m

B1 : Riverbed width (m)
Qsp : Peak discharge of debris flow (630m3/s)
α : 3～5

Target basin area(km²)	α
A≦1.0	2～3
1.0 < A≦10.0	2～4
10.0 < A≦100	3～5

n_{r1}	:	Flow section right bank slope (bottom)	1.00
n_{r2}	:	Flow section right bank slope (top)	1.00
n_{l1}	:	Flow section left bank slope (bottom)	1.00
n_{l2}	:	Flow section left bank slope (top)	1.00
z	:	Debris flow surface water level	
Bda	:	Width of the flow	

Bda= $n_{r1} \times z + n_{l1} \times z$ $(0<z<z1)$
$n_{r1} \times z + n_{l2} \times z$ $(z1<z<z2)$
$n_{r2} \times z + n_{l2} \times z$ $(z2<z)$ ･･･(1)



3.1.2 Debris flow depth (Dd)

Dd= Ad ／ Bda ･･･(2)

Ad : Cross-sectional aera of debris flow peal

Ad= Ad1 $(0<z<z1)$
Ad2 $(z1<z<z2)$
Ad3 $(z2<z)$

3.1.3 Debris flow velocity (U)

$$U = \frac{1}{Kn} \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2} \quad \cdots(3)$$

Kn : Roughness coefficient 0.07
θ : Riverbed surface slope 2.86 (degrees) (i=1／20.0)

3.1.4 Calculation of debris flow velocity and depth

$Q_{special}$ = U ・ Ad ･･･(4)

From formulas (1), (2), (3), and (4), when z is calculated when Qspecial matches Qsp, z is as follows.
z= 1.47m となる。
Calculating debris flow velocity (U) and depth (Dd) from the result of z above, U and Dd are as follows.
Dd= 1.45m , U= 4.27m/s

3.2 Unit weight of debris flow (γd)

γd= (σ×Cd)+{ρ×(1－Cd)}
= {(2,600 × 0.30)+(1,200 × (1－ 0.30)}×9.81/1000
= 15.89 (kN/m³)

3.3 Drag force of debris flow (F)

$$F = \alpha \cdot \frac{\gamma_d}{g} \cdot Dd \cdot U^2$$

$$= 1.0 \times \frac{15.89}{9.81} \times 1.45 \times 4.27^2$$

= 42.83 (KN／m)

3.4 Unit weight of debris flow in water (ρdi)

ρdi=γd－Wo
= 15.89 － 11.77 = 4.12 (kN/m3)

3.5 Unit weight of sediment in water (γs)

γs=C*(σ－ρ)×g/1000
= 0.6 ×(2,600 － 1,200)×9.81/1000 = 8.24 (kN/m3)

※1 Source : SNI 2851-2021_Desain Sabodam

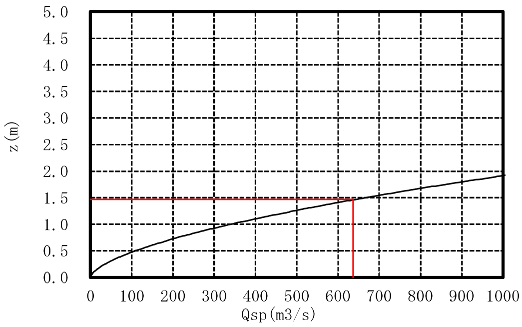
3. 6 Debris flow specifications over Riverbed

Debris flow discharge (Qsp) 630.0 m^3/s
Riverbed surface slope 1/20.0
Riverbed width 100 m

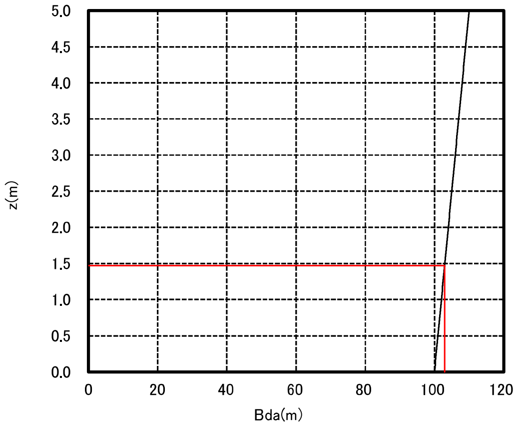
$$Q = 1/n * (Dd)^{2/3} * I^{1/2} * Ad$$

z(m)	B1(m)	Bda(m)	Ad(m2)	Dd(m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
1.22	100.0	102.40	123.49	1.21	0.067	2.86	3.79	468.0
1.23	100.0	102.50	124.51	1.22	0.067	2.86	3.81	474.3
1.24	100.0	102.50	125.54	1.23	0.067	2.86	3.83	480.8
1.25	100.0	102.50	126.56	1.23	0.067	2.86	3.83	484.7
1.26	100.0	102.50	127.59	1.24	0.067	2.86	3.85	491.2
1.27	100.0	102.50	128.61	1.25	0.067	2.86	3.87	497.7
1.28	100.0	102.60	129.64	1.26	0.067	2.86	3.89	504.2
1.29	100.0	102.60	130.66	1.27	0.067	2.86	3.91	510.8
1.30	100.0	102.60	131.69	1.28	0.067	2.86	3.93	517.5
1.31	100.0	102.60	132.72	1.29	0.067	2.86	3.95	524.2
1.32	100.0	102.60	133.74	1.30	0.067	2.86	3.97	530.9
1.33	100.0	102.70	134.77	1.31	0.067	2.86	3.99	537.7
1.34	100.0	102.70	135.80	1.32	0.067	2.86	4.01	544.5
1.35	100.0	102.70	136.82	1.33	0.067	2.86	4.03	551.3
1.36	100.0	102.70	137.85	1.34	0.067	2.86	4.05	558.2
1.37	100.0	102.70	138.88	1.35	0.067	2.86	4.07	565.2
1.38	100.0	102.80	139.90	1.36	0.067	2.86	4.09	572.1
1.39	100.0	102.80	140.93	1.37	0.067	2.86	4.11	579.2
1.40	100.0	102.80	141.96	1.38	0.067	2.86	4.13	586.2
1.41	100.0	102.80	142.99	1.39	0.067	2.86	4.15	593.4
1.42	100.0	102.80	144.02	1.40	0.067	2.86	4.17	600.5
1.43	100.0	102.90	145.04	1.41	0.067	2.86	4.19	607.7
1.44	100.0	102.90	146.07	1.42	0.067	2.86	4.21	614.9
1.45	100.0	102.90	147.10	1.43	0.067	2.86	4.23	622.2
1.46	100.0	102.90	148.13	1.44	0.067	2.86	4.25	629.5
1.47	100.0	102.90	149.16	1.45	0.067	2.86	4.27	636.9
1.48	100.0	103.00	150.19	1.46	0.067	2.86	4.29	644.3
1.49	100.0	103.00	151.22	1.47	0.067	2.86	4.31	651.7
1.50	100.0	103.00	152.25	1.48	0.067	2.86	4.33	659.2
1.51	100.0	103.00	153.28	1.49	0.067	2.86	4.35	666.7
1.52	100.0	103.00	154.31	1.50	0.067	2.86	4.37	674.3
1.53	100.0	103.10	155.34	1.51	0.067	2.86	4.39	681.9
1.54	100.0	103.10	156.37	1.52	0.067	2.86	4.41	689.5
1.55	100.0	103.10	157.40	1.53	0.067	2.86	4.43	697.2
1.56	100.0	103.10	158.43	1.54	0.067	2.86	4.45	705.0
1.57	100.0	103.10	159.46	1.55	0.067	2.86	4.47	712.7
1.58	100.0	103.20	160.50	1.56	0.067	2.86	4.48	719.0
1.59	100.0	103.20	161.53	1.57	0.067	2.86	4.50	726.8
1.60	100.0	103.20	162.56	1.58	0.067	2.86	4.52	734.7
1.61	100.0	103.20	163.59	1.58	0.067	2.86	4.52	739.4
1.62	100.0	103.20	164.62	1.59	0.067	2.86	4.54	747.3
1.63	100.0	103.30	165.66	1.60	0.067	2.86	4.56	755.4
1.64	100.0	103.30	166.69	1.61	0.067	2.86	4.58	763.4
1.65	100.0	103.30	167.72	1.62	0.067	2.86	4.60	771.5
1.66	100.0	103.30	168.76	1.63	0.067	2.86	4.62	779.6
1.67	100.0	103.30	169.79	1.64	0.067	2.86	4.64	787.8
1.68	100.0	103.40	170.82	1.65	0.067	2.86	4.66	796.0
1.69	100.0	103.40	171.86	1.66	0.067	2.86	4.67	802.5
1.70	100.0	103.40	172.89	1.67	0.067	2.86	4.69	810.8
1.71	100.0	103.40	173.92	1.68	0.067	2.86	4.71	819.1
1.72	100.0	103.40	174.96	1.69	0.067	2.86	4.73	827.5

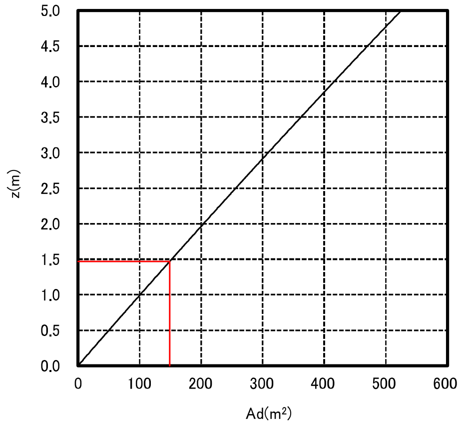
1.47	100.0	102.94	149.16	1.45	0.1	2.86	4.27	636.9



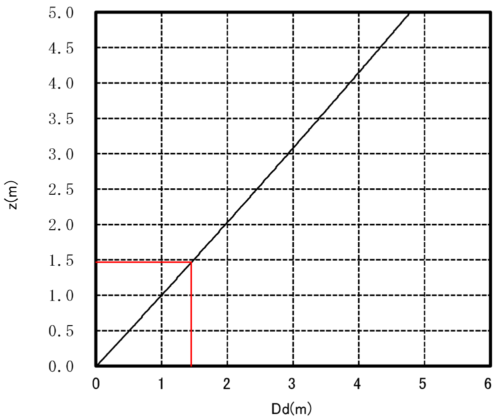
Q_{sp}-Z Curve



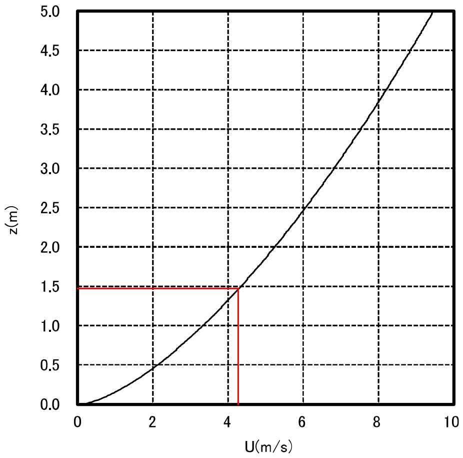
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

4. Height of training dike ※1

4.1 Water depth considering hydraulic jump height

2

The water depth considering the jump height (h_2) is calculated using the following formula.

$$h_2 = \frac{-1+\sqrt{1+8Fr^2}}{2} \times h_1$$

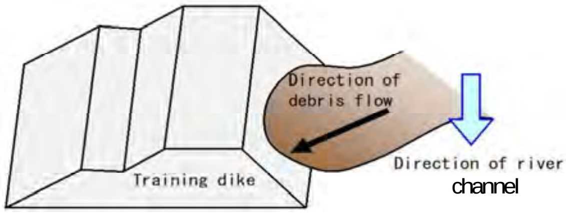
$$= \frac{-1+(1+8\times 1.12\hat{}2)\hat{}(1/2)}{2} \times 1.47$$

$$= 1.71 \text{ m}$$

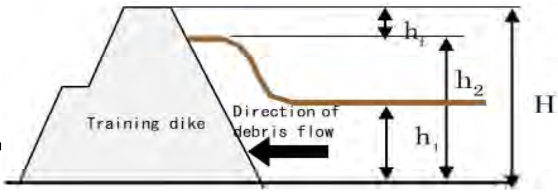
$$Fr = \frac{U}{\sqrt{gh_1}}$$

$$= \frac{4.27}{(9.81\times 1.47)\hat{}(1/2)}$$

$$= 1.12$$



h_1 : Water depth during debris flow (1.47m)
h_2 : Water depth during debris flow + hydraulic jump height
U : Debris flow velocity (4.27m3/s)
Fr : Froude number
g : Gravitational acceleration (9.81m/s^2)



Water depth during debris flow + hydraulic jump height is 1.71m

4.2 Consideration of Training dike height

The height of the training dike is determined by adding the freeboard height to the water depth considering the jump height.
The required freeboard height is shown in the table below.
Since the design discharge is 630m3/s, freeboard height is 1m.

Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6m
200 ～ 500 m^3/s	0.8m
500 m^3/s or more	1.0m

Training dike height is determined according to the following formula.

Water depth during debris flow + hydraulic jump height 1.71m+Freeboard (1m) = **2.8 m**

※1 Source : dT-16-2004-A_Perencanaan Teknis Tanggul pada Sungai Lahar

S4 Area
Tanggul Leprak XVII Kebondeli 2021、Tanggul Leprak II-D、Tanggul Leprak22、23

<u>Training dike</u>

S4 Area
Tanggul Leprak XVII Kebondeli 2021、
Tanggul Leprak II-D、Tanggul Leprak22、23

1. Design condition list

①Topography conditions

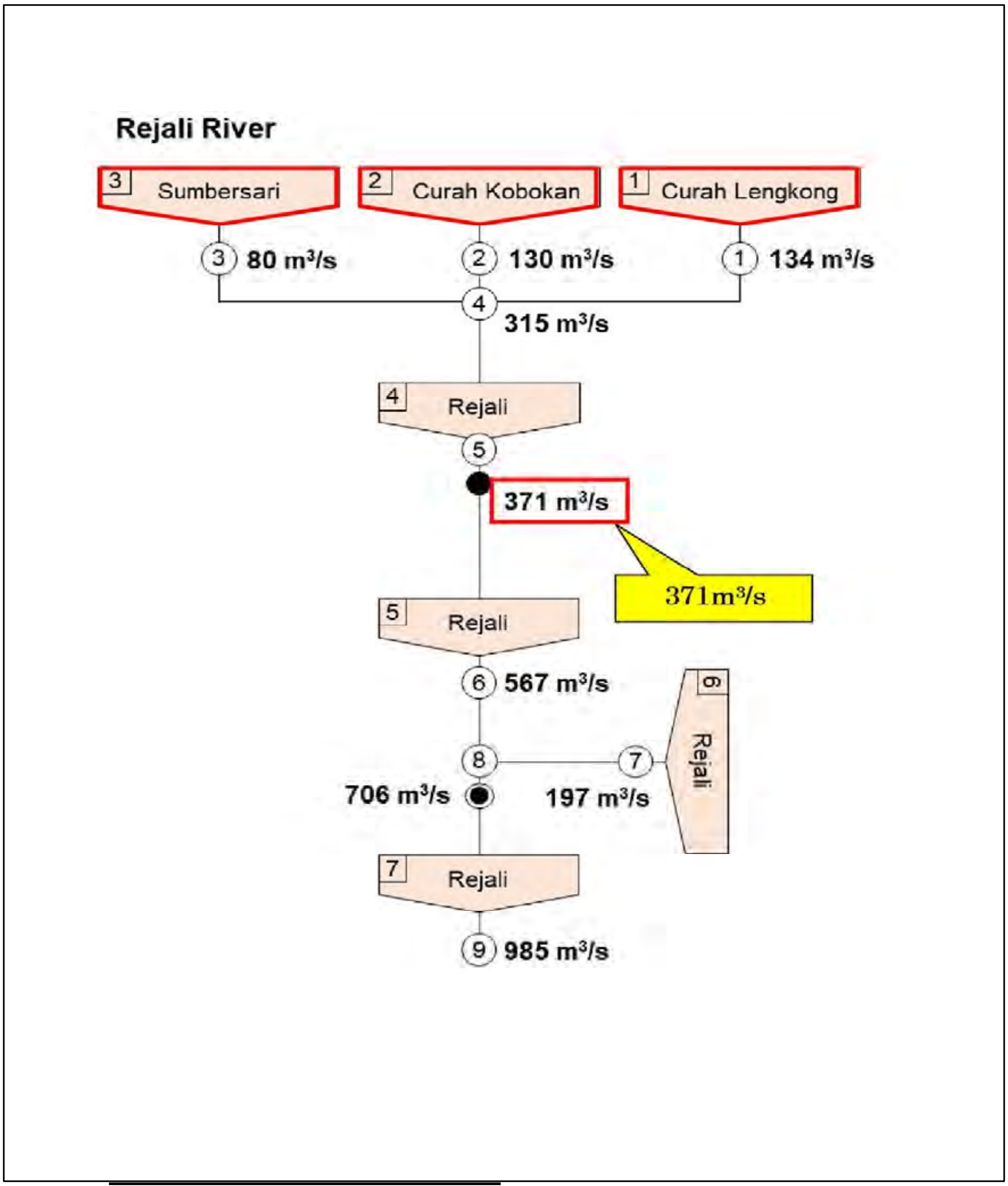
Symbol	Item	Unit	Value
	Topography condition		
θ	Riverbed surface slope		1/22.2
	Flow rate conditions		
A	Target basin area	km^2	19.60

②Design constant

Wo	Unit weight of water	kN/m^3	11.77
σ	Gravel density	kg/m^3	2,600
ρ	Water density	kg/m^3	1,200
K_{p1}	Coefficient		120
C	Coefficient of overflow		0.6
g	Gravitational acceleration	m/s^2	9.81
C_*	Volumetric concentration of deposited sediment		0.6
Kn	Roughness coefficient(front part)		0.067
ϕ	Internal friction angle of sediment	degree	35.0

2. Calculation of design discharge

2.1 Peak discharge of water(Qp)



Therefore, the peak discharge of water without sediment is as follows.

＝ 371 (m^3/s)

2.2 Peak discharge of debris flow(Qsp)

Specifications list

Symbol	Item	Unit	Value
σ	Density of gravel	kg/m^3	2,600
ρ	Density of water	kg/m^3	1,200
Wo	Unit weight of the water	kN/m^3	11.77
ϕ	Internal friction angle of the deposited soil	degrees	35.0
$\tan\theta$	Riverbed surface slope		1/22.2
C_*	Volumetric concentration of deposited sediment		0.60
Kn	Roughness coefficient (the front part of debris flow)		0.07

2.2.1 Concentration of debris flow(Takahashi's Equation)

$$Cd = \frac{\rho \cdot \tan\theta}{(\sigma - \rho)\cdot(\tan\phi - \tan\theta)}$$ ※1

Where,

ρ ; Density of water 1,200.00 (Kg/m^3)

θ ; Riverbed surface slope 2.58 (degrees) (i=1/22.2)

σ ; Density of gravel 2,600 (Kg/m^3)

ϕ ; Internal friction angle of sediment deposited on riverbed 35.0 (degrees)

$$= \frac{1,200 \times \tan 2.58)}{(2,600 - 1,200) \times (\tan 35.0 - \tan 2.58)}$$

= 0.06

When the calculated value (Cd) is larger than $0.9C_*$, Cd is assumed to be equal to $0.9C_*$. Meanwhile, if the calculated value (Cd) is smaller than 0.3, Cd value is assumed to be 0.3

∴ Cd = 0.30

※1 Source : SNI 2851-2021_Desain Sabodam

2.2.2 Calculate Peak discharge of debris flow(Qsp)

The peak discharge of debris flow Qsp (m^3/s) is obtained from the relationship with the peak discharge of water without sediment Qp (m^3/s)

$$Qsp = \frac{C*}{C* - Cd} \times Qp$$ ※1

Where,

Cd : Concentration of debris flow = 0.30

C* : Volumetric concentration of sediment deposited on riverbed = 0.60

Qp : Peak discharge of water = 371 より

$$Qsp = \frac{C*}{C* - Cd} \times Qp = \frac{0.60}{0.60 - 0.30} \times 371 = 742.0$$

Qsp= 742.0 (m^3/sec)

※1 Source : SNI 2851-2021_Desain Sabodam

3. Specifications of debris flow

3.1 Setting of debris flow velocity and depth based on debris flow peak discharge ※1

3.1.1 Debris flow cross-section

The riverbed width (B1) was calculated using the following formula.
For the coefficient α, the median value (α: 4) was adopted.

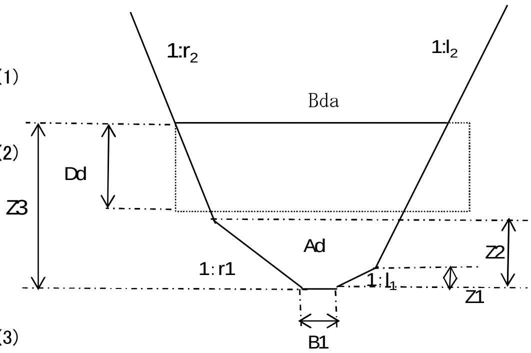
$B1 = \alpha \times Q^{0.5}$
$= 4 \times 742^{0.5} = 109$ m

B1 : Riverbed width (m)
Qsp : Peak discharge of debris flow (742m3/s)
α : 3～5

Target basin area(km^2)	α
A≦1.0	2～3
1.0＜A≦10.0	2～4
10.0＜A≦100	3～5

n_{r1}	:	Flow section right bank slope (bottom)	1.00
n_{r2}	:	Flow section right bank slope (top)	1.00
n_{l1}	:	Flow section left bank slope (bottom)	1.00
n_{l2}	:	Flow section left bank slope (top)	1.00
z	:	Debris flow surface water level	
Bda	:	Width of the flow	

Bda= $n_{r1} \times z + n_{l1} \times z$ (0<z<z1)
$n_{r1} \times z + n_{l2} \times z$ (z1<z<z2)
$n_{r2} \times z + n_{l2} \times z$ (z2<z) ･･･(1)



3.1.2 Debris flow depth (Dd)

Dd= Ad ／ Bda ･･･(2)

Ad : Cross-sectional aera of debris flow pea

Ad= Ad1 (0<z<z1)
Ad2 (z1<z<z2)
Ad3 (z2<z)

3.1.3 Debris flow velocity (U)

$$U = \frac{1}{Kn} \cdot Dd^{2/3} \cdot (\sin\theta)^{1/2} \quad \cdots(3)$$

Kn : Roughness coefficient 0.07
θ : Riverbed surface slope 2.58 (degrees) (i＝1／22.2)

3.1.4 Calculation of debris flow velocity and depth

$Q_{special}$ = U・Ad ･･･(4)

From formulas (1), (2), (3), and (4), when z is calculated when Qspecial matches Qsp, z is as follows.
z= 1.58m となる。
Calculating debris flow velocity (U) and depth (Dd) from the result of z above, U and Dd are as follows.
Dd= 1.56m , U= 4.26m/s

3.2 Unit weight of debris flow (γd)

$\gamma d = (\sigma \times Cd) + \{\rho \times (1 - Cd)\}$
$= \{(2{,}600 \times 0.30) + (1{,}200 \times (1- 0.30))\} \times 9.81/1000$
$= 15.89$ (kN/m^3)

3.3 Drag force of debris flow (F)

$$F = \alpha \cdot \frac{\gamma_d}{g} \cdot Dd \cdot U^2$$

$$= 1.0 \times \frac{15.89}{9.81} \times 1.56 \times 4.26^2$$

$= 45.86$ (KN／m)

3.4 Unit weight of debris flow in water (ρdi)

$\rho di = \gamma d - Wo$
$= 15.89 - 11.77 = 4.12$ (kN/m3)

3.5 Unit weight of sediment in water (γs)

$\gamma s = C*(\sigma - \rho) \times g/1000$
$= 0.6 \times (2{,}600 - 1{,}200) \times 9.81/1000 = 8.24$ (kN/m3)

※1 Source : SNI 2851-2021_Desain Sabodam

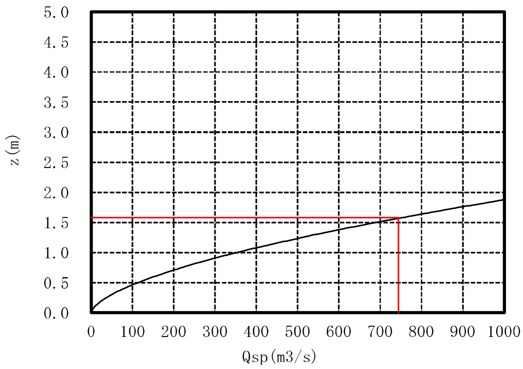
3. 6 Debris flow specifications over Riverbed

$Q = 1 / n * (Dd)^{2/3} * I^{1/2} * Ad$

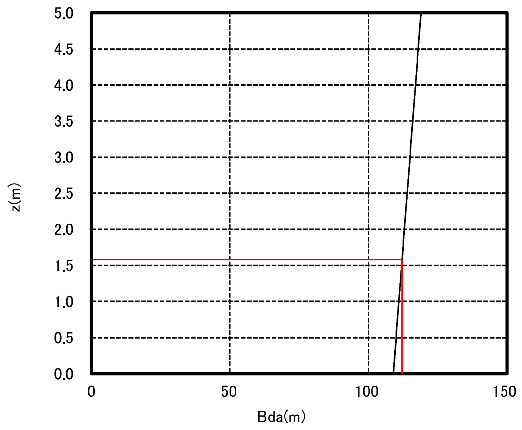
Debris flow discharge (Qsp) 742.0 m^3/s
Riverbed surface slope 1/22.2
Riverbed width 109 m

z (m)	B1 (m)	Bda (m)	Ad (m2)	Dd (m)	n	θ (°)	U (m2/s)	Qsp (m3/s)
1.33	109.0	111.70	146.74	1.31	0.067	2.58	3.79	556.1
1.34	109.0	111.70	147.86	1.32	0.067	2.58	3.81	563.3
1.35	109.0	111.70	148.97	1.33	0.067	2.58	3.83	570.5
1.36	109.0	111.70	150.09	1.34	0.067	2.58	3.85	577.8
1.37	109.0	111.70	151.21	1.35	0.067	2.58	3.87	585.1
1.38	109.0	111.80	152.32	1.36	0.067	2.58	3.89	592.5
1.39	109.0	111.80	153.44	1.37	0.067	2.58	3.91	599.9
1.40	109.0	111.80	154.56	1.38	0.067	2.58	3.93	607.4
1.41	109.0	111.80	155.68	1.39	0.067	2.58	3.94	613.3
1.42	109.0	111.80	156.80	1.40	0.067	2.58	3.96	620.9
1.43	109.0	111.90	157.91	1.41	0.067	2.58	3.98	628.4
1.44	109.0	111.90	159.03	1.42	0.067	2.58	4.00	636.1
1.45	109.0	111.90	160.15	1.43	0.067	2.58	4.02	643.8
1.46	109.0	111.90	161.27	1.44	0.067	2.58	4.04	651.5
1.47	109.0	111.90	162.39	1.45	0.067	2.58	4.06	659.3
1.48	109.0	112.00	163.51	1.46	0.067	2.58	4.08	667.1
1.49	109.0	112.00	164.63	1.47	0.067	2.58	4.09	673.3
1.50	109.0	112.00	165.75	1.48	0.067	2.58	4.11	681.2
1.51	109.0	112.00	166.87	1.49	0.067	2.58	4.13	689.1
1.52	109.0	112.00	167.99	1.50	0.067	2.58	4.15	697.1
1.53	109.0	112.10	169.11	1.51	0.067	2.58	4.17	705.1
1.54	109.0	112.10	170.23	1.52	0.067	2.58	4.19	713.2
1.55	109.0	112.10	171.35	1.53	0.067	2.58	4.20	719.6
1.56	109.0	112.10	172.47	1.54	0.067	2.58	4.22	727.8
1.57	109.0	112.10	173.59	1.55	0.067	2.58	4.24	736.0
1.58	109.0	112.20	174.72	1.56	0.067	2.58	4.26	744.3
1.59	109.0	112.20	175.84	1.57	0.067	2.58	4.28	752.5
1.60	109.0	112.20	176.96	1.58	0.067	2.58	4.30	760.9
1.61	109.0	112.20	178.08	1.59	0.067	2.58	4.31	767.5
1.62	109.0	112.20	179.20	1.60	0.067	2.58	4.33	775.9
1.63	109.0	112.30	180.33	1.61	0.067	2.58	4.35	784.4
1.64	109.0	112.30	181.45	1.62	0.067	2.58	4.37	792.9
1.65	109.0	112.30	182.57	1.63	0.067	2.58	4.39	801.4
1.66	109.0	112.30	183.70	1.64	0.067	2.58	4.40	808.2
1.67	109.0	112.30	184.82	1.65	0.067	2.58	4.42	816.9
1.68	109.0	112.40	185.94	1.65	0.067	2.58	4.42	821.8
1.69	109.0	112.40	187.07	1.66	0.067	2.58	4.44	830.5
1.70	109.0	112.40	188.19	1.67	0.067	2.58	4.46	839.3
1.71	109.0	112.40	189.31	1.68	0.067	2.58	4.48	848.1
1.72	109.0	112.40	190.44	1.69	0.067	2.58	4.49	855.0
1.73	109.0	112.50	191.56	1.70	0.067	2.58	4.51	863.9
1.74	109.0	112.50	192.69	1.71	0.067	2.58	4.53	872.8
1.75	109.0	112.50	193.81	1.72	0.067	2.58	4.55	881.8
1.76	109.0	112.50	194.94	1.73	0.067	2.58	4.56	888.9
1.77	109.0	112.50	196.06	1.74	0.067	2.58	4.58	897.9
1.78	109.0	112.60	197.19	1.75	0.067	2.58	4.60	907.0
1.79	109.0	112.60	198.31	1.76	0.067	2.58	4.62	916.1
1.80	109.0	112.60	199.44	1.77	0.067	2.58	4.63	923.4
1.81	109.0	112.60	200.57	1.78	0.067	2.58	4.65	932.6
1.82	109.0	112.60	201.69	1.79	0.067	2.58	4.67	941.8
1.83	109.0	112.70	202.82	1.80	0.067	2.58	4.69	951.2

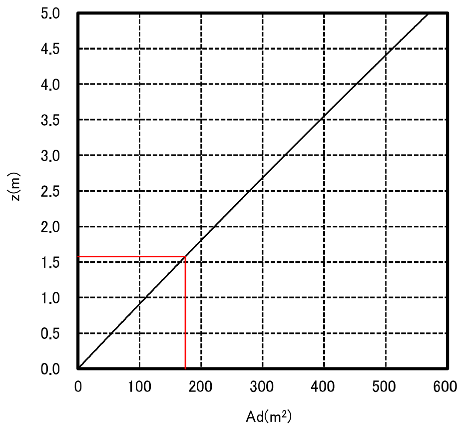
1.58	109.0	112.16	174.72	1.56	0.1	2.58	4.26	744.3



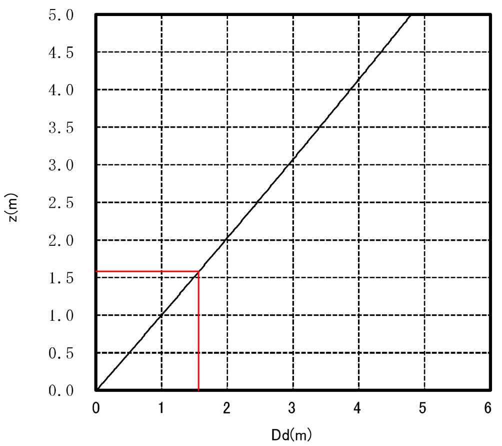
Q_{sp}-Z Curve



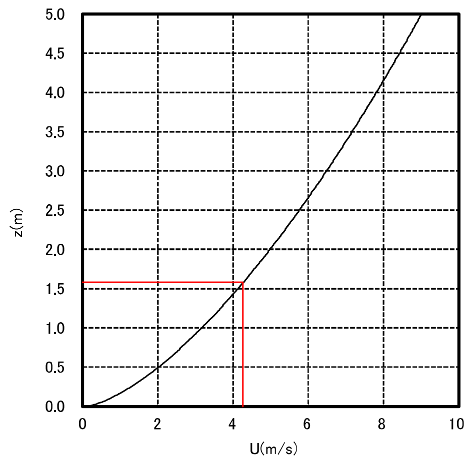
B_{da}-z Curve



A_d-Z Curve



Dd-Z Curve



U-Z Curve

4. Height of training dike ※1

4.1 Water depth considering hydraulic jump height

The water depth considering the jump height (h_2) is calculated using the following formula.

$$h_2 = \frac{-1+\sqrt{1+8Fr^2}}{2} \times h_1$$

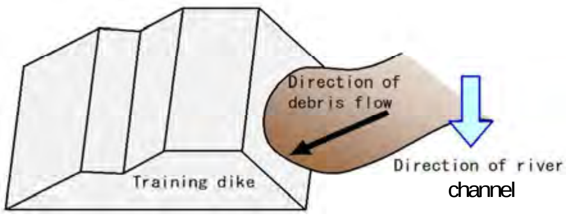
$$= \frac{-1+(1+8\times1.08\hat{}2)\hat{}(1/2)}{2} \times 1.58$$

$$= 1.75 \text{ m}$$

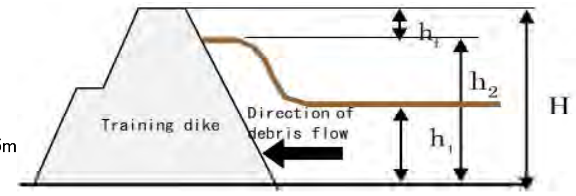
$$Fr = \frac{U}{\sqrt{gh_1}}$$

$$= \frac{4.26}{(9.81\times1.58)\hat{}(1/2)}$$

$$= 1.08$$



h_1 : Water depth during debris flow (1.58m)
h_2 : Water depth during debris flow + hydraulic jump height
U : Debris flow velocity (4.26m3/s)
Fr : Froude number
g : Gravitational acceleration (9.81m/s^2)



Water depth during debris flow + hydraulic jump height is 1.75m

4.2 Consideration of Training dike height

The height of the training dike is determined by adding the freeboard height to the water depth considering the jump height.
The required freeboard height is shown in the table below.
Since the design discharge is 742m3/s, freeboard height is 1m.

Freeboard height

Design discharge	Freeboard height
Less than 200 m^3/s	0.6m
200 ～ 500 m^3/s	0.8m
500 m^3/s or more	1.0m

Training dike height is determined according to the following formula.

Water depth during debris flow + hydraulic jump height 1.75m+Freeboard (1m) = **2.8 m**

※1 Source : dT-16-2004-A_Perencanaan Teknis Tanggul pada Sungai Lahar