Japan International Cooperation Agency (JICA)

Oromia Irrigation Development Authority (OIDA)

Technical Guideline for Design of Irrigation Canal and Related Structures

May, 2014

The Project for Capacity Building in Irrigation Development (CBID)

Foreword

Oromia Irrigation Development Authority (OIDA) is established on June, 2013, as a responsible body for all irrigation development activities in the Region, according to Oromia National Regional Government proclamation No. 180/2005. The major purposes of the establishment are to accelerate irrigation development in the Region, utilize limited resources efficiently, coordinate all irrigation development activities under one institution with more efficiency and effectiveness.

To improve irrigation development activities in the Region, the previous Oromia Water Mineral and Energy Bureau entered into an agreement with Japan International Cooperation Agency (JICA) for "The Project for Capacity Building in Irrigation Development (CBID)" since June, 2009 until May, 2014. CBID put much effort to capacitate Irrigation experts in Oromia Region through several activities and finally made fruitful results for irrigation development. Accordingly, irrigation projects are constructed and rehabilitated based on that several Guidelines & Manuals and texts produced which can result in a radical change when implemented properly.

Herewith this massage, I emphasize that from Now on, OIDA to make efforts to utilize all outputs of the project for all irrigation activities as a minimum standard, especially for the enhancement of irrigation technical capacity.

I believe that all OIDA irrigation experts work very hard with their respective disciplines using CBID outputs to improve the life standard of all people. In addition, I encourage that all other Ethiopian regions to benefit from the outputs.

Finally, I would like to thank the Japanese Government, JICA Ethiopia Office, and all Japanese and Ethiopian experts who made great effort to produce these outputs.

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Introductory Remarks

"Growth and Transformation Plan" (GTP) from 2011 to 2015 intensifies use of the country's water and other natural resources to promote multiple cropping, better adaptation to climate variability and ensure food security. Expansion of small scale irrigation schemes is given a priority, while attention is also given to medium and large scale irrigation.

In Oromia Region, it is estimated that there exists more than 1.7 million ha of land suitable for irrigation development. However, only 800,000 ha is under irrigation through Traditional and Modern irrigation technology. To accelerate speed of Irrigation Development, the Oromia National Regional State requested Japan International Cooperation Agency (JICA) for support on capacity building of Irrigation Experts under Irrigation Sector.

In response to the requests, JICA had conducted "Study on Meki Irrigation and Rural Development" (from September 2000 to January 2002) and Project for Irrigation Farming Improvement (IFI project) (from September 2005 to August 2008). After implementation of them there are needs to improve situation on irrigation sector in Oromia Region.

JICA and the Government of Ethiopia agreed to implement a new project, named "The project for Capacity Building in Irrigation Development" (CBID). The period of CBID is five years since June, 2009 to May, 2014 and main purpose is to enhance capacity of Irrigation Experts in Oromia Region focusing on the following three areas, 1) Water resources planning, 2) Study/Design/Construction management, 3) Scheme management through Training, On the Job Training at site level, Workshops, Field Visit and so on and to produce standard guidelines and manuals for Irrigaiton Development.

These guidelines and manuals (Total: fourteen (14) guidelines and manuals) are one of the most important outputs of CBID. They are produced as standards of Irrigation Development in Oromia Region through collecting different experiences and implementation of activities by CBID together with Oromia Irrigation Experts and Japanese Experts.

These guidelines and manuals are very useful to improve the Capacity of OIDA Experts to work more effectively and efficiently and also can accelerate Irrigation Development specially in Oromia Region and generally in the country.

Finally, I strongly demand all Irrigaiton Experts in the region to follow the guidelines and manuals for all steps of Irrigation Development for sustainable development of irrigation.

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1. GENERAL DESCRIPTION

1.1 Aim of the Manual

This manual discusses general terms to be considered in the design and construction or rehabilitatiom of canals in irrigation projects.
The manual defines the general and basic technicals terms related to the standards design and construction of canals. Therefore, design and construction of each canals in different conditions, technical and economic criteria must be considered according to the principles set out in this manual, depending on the canal function, network, system, scale and other field conditions such as topography, etc.

1.2 Scope of the Manual

This manual is applied to the design and construction of canals with main purpose of irrigation and drainage of agricultural land particilarly for canals which are designed and constructed in the standard scale and conditions.

1.2.1 Classification of Canals and Application

A canal is a series of structures (including facilities for water distribution, water measurement, confluence, etc) to convey a required amount of water from one place to another with a certain purpose. Canals are broadly divided into canals for conveying water (such as main canals and lateral canals) and canals for distributing or collecting water (such as farm ditches and farm drains), according to their functions and hydraulic characteristics. This manual mainly deals with canal system to flow irrigation water and drained water of agricultural land.

However, of the above canals, special canals such as water warming canals, muddy water canals, water way-roads, village water supply and village sewage canals are excluded from this manual due to the scope in principle.

1.2.2 Fluid

The fluid considered in this manual is ordinary fresh water that is usable as irrigation water, and water of ordinary quality such as rainwater, groundwater, etc. in agriclutural lands. Special measures are required to select materials for canals and to determine canal stsructures, if warm water, liquid chemical, liquid fertilizer, waste water from livestock and domestic activities, etc, are contained.

1.3 Classification of Canals

In this manual, canals are classified by their purpose of use, system and type as follows:

-Classification by purpose: Irrigation canal, drainage canal, dual-purpose canal,
-Classification by system : Main canal, lateral canal,
-Classification by type : Open canal, pipeline,

1.3.1 Classification by Purpose

(1) Irrigation canal

Irrigation canal is a canal to convey agricultural water mainly irrigation water. It is divided into the exclusive irrigation canal for conveying irrigation water only and the multipurpose canal for convying city water, industrial water or power generation water together with irrigation water.

(2) Drainage canal

Drainage canal is a canal to drain mainly surface water and groundwater from agricultural land and village sewage. It includes the canal to drain excess water from agricultural land in order to secure the growth of crops and to mechanize farming works, and the canal for agricultural land conservation by protecting agricultural land from water erosion, etc.

(3) Dual-purpose canal (for irrigation and drainage)

In principle, irrigation canals and drainage canals must be separated from each other. However, in upland fields on sloping land, the following canals are used as dual-purpose canals: 1) canals which are used for irrigation during the irrigation season and for drainage during the flood season, 2) canals which are used as drainage canals for upland fields in higher area and as irrigation canals for those in lower area. These canals are generally designed so as to have an adequate capacity for drainage discharges. In the case of 1), the irrigation water level must be periodically checked during the irrigation season, and special attention must be paid to the canal management for drainage.

1.3.2 Classification by System

The irrigation canal is classified into the main canal that conveys irrigation water from the intake point to the irrigation areas, and into the lateral canal, which is branched off from the main canal to distribute water into individual irrigation blocks. The drainage canal is also classified into the main canal and lateral canal from the drainage outlet to the upstream. The main irrigation canals, canals which convey irrigation water from the intake points to the regulating reservoirs, major division works, etc. are often called head races. Lateral canals are also divided into secondary and tertiary canals. In addition to the classification of canal systems by function mentioned above, there are also catch drains that are drainage canals constructed along a contour line at the boundaries between the foot of a mountain and pumped drainage areas or gravity drainage areas to intercept drainage water from high lands. Catch drains mitigates flood damages in the low lands and outlet channel, which are used to discharge all or part of the flood water to rivers, lakes, or sea for flood control, removal of sand and maintenance of canals.

1.3.3 Classification by Type

In this manual, canals are classified into open channel and pipeline according to the hydraulic and structural characteristics of canals.

(1) Open channel type

Open channel type is a canal having a free water surface without hydraulic pressure. It includes open channels, tunnels, culverts, siphons, etc. Even if a canal system partially includes closed conduits under hydraulic pressure such as siphons, etc., it is still called an open channel type as a whole.

(2) Pipe line type

Pipe line type is mainly composed of pipelines receiving internal hydraulic pressure without a free water surface. Structurally, it is divided into the closed type of which all lines are composed of closed conduits and the open type which comprises stands having a free water surface on the way of the line at the end.

1.4 Canal System

The canal system consists of water conveying facilities, diversion facilities, water measuring facilities, canal junction facilities, regulating facilities, protection and safety facilities, operation and maintenance facilities, and appurtenant facilities. A canal is composed of various facilities which are systematically connected with each other to organize the canal system and fulfill the functions of the canal as a whole. Classification of facilities and their functions are as follows:

1.4.1 Water Conveying Facilities

This is the main facility of the canal system for conveying water. It is divided into open channel and pipeline depending on the topography of the canal route, land use, hazards and other field conditions. Further, the open channel type consists of open channel, tunnel, culvert, aqueduct, siphon, drop, chute, etc.

1.4.2 Diversion Facility, Water Measuring Facility and Canal Junction Facility

The division facilities are for regulating and distributing irrigation water from the main irrigation canal to lateral canals, or from lateral canals to farm ditches depending on the required amount of irrigation water. The measuring facility is provided to measure and record the discharge for effective use of irrigation water. It is generally combined with the division boxes to check the divided discharge and to distribute water rationally. The canal junction facility is constructed mainly in the drainage canals to lead the drained water from the lateral canal to the main canal or from farm drains to the lateral canal. It consists of junction works, drop works, etc.

1.4.3 Regulating Facilities

For securing the function and safety of the canal, there are regulating facilities for the water level and discharge, spillways, wasteways, pressure regulating facilities, drainage gates, etc., which are used to regulate the water level, pressure, velocity and discharge in the canal. There are also regulating the discharge or timely variation of the water level in the irrigation and drainage system in order to use water effectively, to maintain the drainage function, to give flexibility to the canal functions and to rationalize the canal facilities.

1.4.4 Protection Facilities

In order to protect the canal facilities functionnaly and structurally, there are cross-drainages, drainage inlets, drainage ditches, sedimentation tanks, slope protection works, etc. Spillways and wasteways equallyclassified as protection facilities, but they are classified as regulating facilities in this manual.

1.4.5 Safety Facilities

These are facilities for ensuring the safety of supervisors of the canal system and others that include guardrails, fences, handrails, life ropes, ladders, signs, etc.

1.4.6 Operation and Maintenance Facilities

These are facilities for water management and operation and maintenance of canal facilities, which include observation facilities, control facilities, communication facilities, control offices, operation and maintenance roads and dust removal facilities, etc.

1.4.7 Appurtenant Facilities

These are compulsory facilities necessary for maintaining the functions of existing facilities owing to construction or rehabilitation of canals. Bridges and crossing structures are included in this category.

1.4.8 Other Related Facilities for Water Utilization

The canal is a comprehensive irrigation and drainage system to fulfill its function for water utilization, which includes the other related facilities for water utilization such as dams, headworks, pumping stations, etc., and rivers, lakes and sea as water resources or drainage outlets. Therefore, in designing canals and in planning water management, these related facilities mentioned above must be considered as the components of series of comprehensive irrigation and drainage system.

1.5 Basic Considerations in Canal Design

In designing canals, the following matters must be considered basically. The canal system must keep the necessary functions for water utilizations as a series of irrigation and drinage facilities. It must be designed as an economical system in additions to be a safe and rational system for water management. At the same time, the design must comply with relevant rules and regulations.

1.5.1 Basic Considerations in Design

The design of canals must be done so as to fulfill their functions for irrigation and drainage efficiently for a safe and rational management of water use and facilities. It helps also to minimize construction costs as well as operation and maintenance costs.

(1) Information necessary for design

In designing canals accurate information must be collected on the followings points:

1) Purpose of canal system

The purpose of canal system according to the use of canal, such as for upland field irrigation, flood drainage and normal drainage.

2) Irrigation and drainage water requirements, water level, etc.

The reasons for requirements, period and place of requirements of irrigation and drainage, required water level, etc,

3) Present conditions of commanded area by canals

The range of commanded areas, topography, geology, climate, hydrology, land use, farm management, irrigation and drainage networks, customs, etc,

4) Water resources and drainage outlets

The type, scale, location, discharge, water level and river condition of water sources or drainage outlets such as dams, head works, pumping stations, rivers, lakes, seas, etc,

5) Present conditions of canal routes

The topography, geology, land use and other rights along the canal routes,

6) Operation and maintenance

The organization, method, costs, etc. of operation and maintenance after completion of canal facilities.

(2) Basic considerations in design of canals

The design of canals must be done so as to provide an economic canal system with special attention to fulfill their functions for conveying and distributing water efficiently. It is as a comprehensive irrigation and drainage system in combination with other related facilities for water use, and to make possible safe and rational management of water use and facilities. In this respect, it is basically considered in design to keep 1) water conveyance capacity, 2) water distributing, confluence and regulating functions, 3) safety of the canal, 4) rational management of water use and facilities, 5) economic cost for construction and operation and maintenance, and 6) harmony with the surrounding environments.

1.5.2 Consideration of Rules Related to Canal Design

Canals are long-term structures that connect to rivers and lakes and are provided over a vast area. In design of canals related rules and regulations must be considered.

1.6 Basic Considerations in Canal Construction

The canal must be constructed according to the plan prepared for undertaking the work rationally, economically and safely, as well as to satisfy design details, in consideration of the field conditions. At the same time, the canal construction must comply with the relevant rules and regulations.

1.6.1 Basic Consideration in Canal Construction

The construction must be executed economically and safely within the proposed period according to the design. Therefore, planning the construction must be prepared taking into account the intention of design and field conditions and the progress of the works must always be checked. If any field conditions different from design conditions are encountered, then the design must be restudied and modified.

1.6.2 Construction

Construction must be executed rationally, economically and safely in accordance with an appropriate construction planning and under construction management in consideration to field conditions.

2. BASIC DESIGN INPUT DATA (INVESTIGATION)

2.1 Basic Consideration in Investigation

Investigation must be executed according to well-conceived plan, by an appropriate methods and using standard procedures to obtain fundamental data that are necessary for route selection, design, construction, operation and maintenance of canals. Investigations are required to collect the necessary basic data for determination of canal routes, selection of canal types and facility designs, planning of construction methods and future management of canal system, after proper recognition of the purpose and site conditions for the canals. Investigation must, therefore, be executed and controlled systematically from the start by an experienced engineer with knowledge in planning of canals including design, construction, operation and maintenance, etc.

2.1.1 Plan of Investigation

In order to collect the data required for each stage in canal construction, investigations must be made according to a carefully established plan for their scope, methods, precision and other conditions.

(1) General items

Canal construction generally proceeds through planning, investigation, design and construction. As a general rule, the outline of investigation is determined at first and then the details are studied. Investigation of canal works varies in the terms, range, principle, method and precision depending on the stage reached in canal construction. Therefore, the investigation must proceed with careful planning. Investigation for canal works may include ① those necessary for planning, ② those necessary for design, ③ those necessary for construction, ④ those necessary for operation and maintenance, and ⑤ others. Methods of investigation may include ① data collection, ② reconnaissance survey, ③ field investigations, surveys, field tests, field observations, ④ laboratory tests, ⑤ trial construction, observation after construction, and ⑥ supplementary tests, etc. Investigation for the route selection, designing and construction of canals do not merely involved technical problems, but touch on socio-economics and environmental issues in the broadest sense. Such investigation may take a long time from the start to completion, and various corrections may be required in the process due to the interaction of the results. Hence, it is difficult to divide such investigation into definite steps. In general, they may be divided into ① Identification, ② Pre-feasibility study, ③ Feasibility study and ④ supplementary investigations complementary to execution of the construction.

(2) Steps of investigation

1) Identification

The objective of identification is to evaluate broadly the existing data on weather, hydrology, topography, geology, and site conditions. It is also to reconnoiter the site and make other necessary studies with a view to establishing the basic plan of the canal route. Usually, several alternative routes are compared, and two or three routes are proposed in plans area set up.

2) Pre-feasibility study

Pre-feasibility study is made to provide data for the basic designs that include the selection of the final canal route, basic design of facilities, basic planning of construction and broad estimate of cost. Pre-feasibility consists of data collection, topography survey and soil tests. Usually in this step, the final plan for canal construction is determined. Although the details may be revised later as the result of further field investigations, sufficient investigations are necessary to ensure that the basic plan will not be drastically changed later.

3) Feasibility study

Feasibility study is made to collect data necessary for detailed designs such as the detail design of facilities, detailed cost estimate and construction planning on the basis of the basic design. These investigations are required to supplement previous investigations in both quality and quantity of detailed design. It will usually be based on the results of previous investigations for the identification and pre-feasibility.

4) Supplementary investigation

These investigations are to obtain supplementary data during the construction stage if found to be necessary. Ideally, the design and construction plan will be fully determined as the result of detailed preliminary investigations, but if actual site conditions prove to be significantly different from those assumed in the design or if any accident would occur or if changes in construction or sitting are otherwise required for any material or social reason, suitable supplementary investigation will be required.

2.1.2 Items for Investigation

Items for investigation necessary for the design, construction and maintenance of canals must be determined in each case according to the purpose of the canals, site conditions and other factors, so that investigations may be carried out rationally and efficiently. Items for investigation necessary for canal construction should be decided at first, and investgated according to procedures and methods appropriate to each case. Typical items relating to each matter are listed below. The items necessary for construction of a specific canal are selected from this list and are investigated rationally and efficiently.

(1) Investigation items for overall plan

These are the items necessary to prepare the project planning, which is the basis of the design. They are mainly taking into account the internal and external situation of the project area, relationship with other projects and any local plans, and also to confirm the information obtained from the existing data.

1) Project area,
2) Existing irrigation and drainage systems,
3) Existing irrigation and drainage facilities,
4) Customs of irrigation and drainage,
5) Long –range prospect of water demand,
6) Possibilities for rationalization of agricultural water use,
7) Present land use and capability,
8) General social, economic and agricultural factors to be considered (Including institutional factors),
9) River conditions,
10) Relation with other projects,
11) Local development plan.

(2) Investigation items for design and construction

There are items for obtaining the nescessary fundamental data for practical determination of the design and construction plan. The items are mainly intended to ensure complete understanding of the natural and site condition.

1) Topography,
2) Soil conditions (especially engineering soil properties and permeability), geology,
3) Meteorology, hydrology (temperature, rainfall, water level, discharge, river conditions, groundwater level, etc.),
4) Site condition (social condition, construction condition, environmental condition).

(3) Investigation items for operation and maintenance

These are the items necessary for determining the method of operation and maintenance and its maintenance facilities after the canal construction, and are intented to investigate the circumstances in similar local areas, control and other devices, mainly relating to the operation and maintenance system in the future.

1) Meteorology, hydrology,
2) River conditions,
3) Basic data on observation, control, communications, and registration facilities,
4) Water control system and control data of existing canals,
5) Operation and maintenance system and its level after completion of canal,
6) Road condition, traffic volume, etc.

(4) Other investigation items

These items may include investigations relating to land acquisition for the canal construction, its compensation, and harmony with natural environments and living environments.

1) Compensation investigations
2) Environmental investigations

2.2 Investigations

2.2.1 Topographical Investigations and Surveys

Topographical investigations and surveys include the collections of topographical data in the whole project area, preparation of topographical maps and topographical surveys along the proposed canal routes. Topographic surveys should be started from the collection of existing data, and promoted from rough to fine surveys.

(1) Data collection

The following maps published by government are used:

1) Topographic maps (scale 1/25,000 or 1/50,000)
2) National topographic series (scale 1/2,500 or 1/5,000)
3) Land use maps (Land Classification Maps)
4) Detail Maps (scale 1/1,000-1/10,000)

(2) Aerial photographic survey

The topographical maps used for desk study can be modified as project area maps from various published topographical maps. If published maps are inadequate, then topographical maps (scale 1/2,500 to 1/5,000) should be compiled with reconnaisance and verification survey. Recently they are mainly compiled by aerial photography. The scale of aerial maps must be determined according to the purpose of use and land situation, and the density and distribution of datum points will be determined acording to the required and proposed uses for the maps. In this regards, the features of aerial photographic survey are as follows:

1) Precision of all sations is uniform,
2) All stations are surveyed nearly simultaneously,
3) The land can be surveyed without field work,
4) Not suited to large scale such 1/100 to 1/400,
5) The cost is relatively high for a small area.

(3) Route survey

Routes are selected approximately on the drawing board based on available maps and aerial photographs, and ground survey is effeted with respect to the selected routes. This ground survey is the actural survey in the field to obtain data for basic design and the detailed design. (Refer Table 2.1)

1) Central line survey

Central line survey is the survey to locate the center line of the canal on the ground. Center stakes are driven from the starting point as specified intervals and precision, and are numbered sequentially from the starting point. Additional stakes are placed where necesary.

2) Longitudinal survey

Longitudinal survey is to measure the stake levels of survey points and additional points set on the center line in order to plot a longitudinal section along the center line. The longitudinal section is filled in with proposed water levels and canal bed. It is an important survey drawing used in canal system design and facility designs. Bench marks (B.M.) used for reference are placed at specified intervals along the route.

3) Cross section survey

Cross sections are plotted for a center stake positions. These sections are filled in with the sections of excavation and embankment, and are used in canal system designs and facility design.

4) Plane-table survey (Toporaphic Survey)

The plane-table survey is usded to measure and plot the land topography and boundaries. The chart made by this survey is completed with the names of places and structures to provide a map along the canal route. This map is used in the design of facilities and planning of construction. At proposed important structures the necessary extent and scale must be determined individually.

5) Land survey

In land surveying, stakes are placed along the center line to make the limits of land ownership. This map is used in land compensation.

Table 2.1 Standard of drawing of survey maps

Survey maps	Method of survey	Scope of survey	Scale	Contour interval	Survey station interval	Description
Topograph-ic map	Aerial photograph	All the related Region	1/5,000 ~1/2,500	2.0~1.0m		For convenience, profile map/ Horizontal length should be plotted on the topographic (plane) map with the same scale. For drainage canal, contour lines of at least 0.5m are needed in order to obtain data about depth, volume and area of pending.
Plane map Profile map Section map	Route survey Center line, survey longitudinal and cross section survey plan survey	One side of route Approx. 30~100m	1/1,000 ~1/200 1/500 ~1/100 1/200 ~1/100	Irrigation canal 1.0m Drainage canal 0.5m	Approx. 50~100m	
Structure plane map	Plain survery for structures		1/200 ~1/50	Irrigation canal 1.0m Drainage canal 0.5m		
Land survey map	Land survey					

2.2.2 Soil and Geological Investigations

Soil and geological investigations including the collection of geological data reconnaissance, auger holes, trail pits or borings are made along the proposed canal routes to understand the geological structure, the physical properties of the soil, the groundwater table and other conditions.

(1) Items and methods of investigations

Soil and geological investigations are important studies for considering the basic design and construction such as canal route, type of canal or structure, and method of construction. The investigaitaon should be made rationally with respect to the items required depending on the purpose of determination of route, selection of type, etc., from the results of paper study and field reconnnaissance, at each step of ① Identification, ② Pre-feasibility study, ③ feasibility study and ④ supplementary investigations. The content of these investigations may include the items listed below. The method of investigation should be selected with reference to the following matters depending on the scale of the canal, degree of importance, and the quality of soil.

1) Geophysical exploration

This is suited to geological investigation of relatively shallow areas and it is divided into surface exploration method and inter-hole exploration methods. Earthquake exploration is often used among surface exploration methods and sometimes electrical exploration (velocity exploration and PS exploration) among inter-hole exploration methods. When we use these methods, we should understand their adaptabilities and limits, and consider the results after putting together each informations.

2) Sounding

This is the method of exploring the properties of the soil bed from the resistance of penetration, rotation, and withdrawal of an inserted resistance. Not only the geological structure, but also the properties of the soil may be indirectly estimated from the test values. These investigations are often conducted simultaneously with other explorations (drilling). The standard penetration test is a common method, but the cone or Swedish sounding method is economical in soft ground, and it is suited to a wide range of investigation.

3) Drilling by auger, shell, or rotary method

This is a method of sampling soil or rocks from the underground.It is also used for other field tests or inserting instruments by using the drilled hole. While the geophysical exploration or sounding are indirect methods of investigating soil conditions, drilling is the common method of obtaining representative samples and undisturbed samples directly and continuously, and this is very important because the data obtained can be used directly in the design or construction planning.

4) Test pitting

By digging test pits or trenches manually or mechanically, soil quality may be observed directly, and disturbed or undisturbed samples may be obtained. This is the most reliable method of subsurface investigation, but it is not economical for investigation of deep layers. Test pits may be excavated either vertically or horizontally, and attention must be paid to the safety condition during and after digging, by supporting timbering or fence, depending on the size of pits.

5) Groundwater investigation

These are mainly intended to measure groundwater level and test permeability of the sub grade. For measuring groundwater levels, the bore holes or well observation may be used the field water permeability test is intended to determine the permeability of the ground, and various methods are available as shown below. Water injection may be used to obtain the coefficient of permeability by injecting water into a bore hole, auger hole, etc where the ground water level is low. Pumping out of a well or bore hole may be applied where the groundwater level is high, to find the permeability coefficient of the water bearing layer by observing the recovery of water level after lowering by pumping. The component of this method is the combination of a pumped well and observation well(s). Future tracer's methods may be used to investigate the actual flow of underground water or of water leaking by injecting a dye or electrolytic substance into a bore hole or vertical hole, and detecting it in other bore hole, shaft, adit or at the surface water outlet.

6) Investigations of bearing capacity or deformation of ground

For these investigations, it is preferable to obtain dynamic constants relating to bearing capacity and deformation directed by a loading test. In certain cases however, the necessary data may be estimated from other test results (such as standard penetration tests, shearing tests, etc.) Loading test methods include the plate loading test to apply load in the vertical direction by using a plate, and horizontal direction ground loading test to apply load to the side wall of bore holes or test pits.

7) Soil test

Roughly soil tests can be classified as physical properties tests, mechanical tests, and chemical properties tests. In the investigation, the target soil is classified by physical properties tests according to the Unified Soil Classification System (details are mentioned later). Physical properties tests are used to estimate the soil classification made by former data and soil properties and to calculate fundamental properties of soil (void ratio, degree of saturation, density, etc.). Mechanical tests may also be depending on the objective of the investigation. Since undisturbed samples are used in the tests of bearing capacity and physical properties, while disturbed samples are necessary for construction materials. It is necessary to consider the sampling method in the plannig of boring or test pit investigation. Chemical properties tests and rock tests may also be required. The principal items of soil testing and use of their results are shown in the Table 2.5.

Table 2.2 Sounding suitable for Soil and strength of Ground
(Soil survey method, arranged by Soil Engineering Institute)

Ground	Suitable sounding	Remarks
(1) Unknown ground	Standard penetration test	Data of soil quality and strength are obtainable at the same time. It has a large exploration capability and is most suitable as initial surveying means.
(2) Ground mainly comprising sand and pebbles (regardless of value)	1. Standard penetration test 2. Large-scale dynamic penetration test	Single tube sounding without combining with boring may be degraded in performance due to skin friction
	1. 10 t Dutch cone 2. Swedish sounding	
(3) Ground with alternating sand and silt layers of medium or more strength and clay 4<N<30	1. Standard penetration test, large-scale dynamic penetration test. 2. Borro dynamic Penetration test	(1) is suitable for deep exploration, and (2) for shallow one.
	1. 10 t Dutch cone. 2. Swedish sounding, 2t Dutch cone	
(4) Ground with silt and clay of medium or less strength 2<N<4	1. Borro dynamic Penetratioin test	(1) is suitable for deep exploration, and (2) for shallow one. Single tube souding may cause skin friction.
	1. 2 t Dutch cone, Swedish sounding 2. Portable cone (double tube), "pane" (in hole)	
(5) Ground with clay, silt and peat of extreme weakness N<2	1. Portable cone (single tube, double tube) 2. "Pane ", Isky meter"	Static test machine should be small size-needs adjustment of and also it is necessary to adjust dead weight of rod as well as local friction.

Table 2.3 Classification of Boring Surveying Objective and Sampling Method-Type of Boring as Surveying method

(Soil surveying method, arranged by Engineering Institute)

Name of method	Objective	Sampling tool	Quality and use of sample	Applied boring method
Auger boring	Examine structure of unconsolidated ground, approximate water content of soil and ground water.	Soil auger	Considerably disturbed sample. For classification and arranging specimen of soil .	Hand auger boring. Machine auger boring.
Drivie sample boring	Examime structure, thickness, depth and layers of soil, foundation condition and charactreristics as borrow material except for hard rock pebbles.	Driving/thrusting sampler for “drive sample” drive barrel (spit spoon)(standard penetration test tool) and others.	Representative sample though disturbed. Classification water content measurement and arranging specimen of soil.	Machine auger boring, rotary boring, wash boring, displacement boring and pile boring.
“Undisturbed” sample boring	Collect cohesive soil which indicates (in tests) properties similar soil in the original condition. Also to carry out survey of dynamic properties of cohesive soil.	Sampling tool for collecing “undisturbed” samples by thin-wall tube and others.	Little disturbed sample similar to original soil. For classification., water content measurement. Shear test, consolidation test, and arranging specimen of soil.	Machine auger boring, rotary boring, foil sampling
Core boring	Collect continuous rock core samples.	Coring bit with core tube barel, diamond or alloy	Undisturbed or little disturbed rock core. For compression test, tensil test, hardness test, and arrenging specimen of soil.	Rotary (core) boring
Trial excavation survey	Collect most undisturbed samples and also make direct observation of original ground. It is a special survey for serious cases of soil stability and permeability, and for large quantities of test samples.	Manual block sampling, box sampling, preceeding trimming sampling shovel.	Sample, least disturbed sample. For classification, water content measurement, shear test, consolidation test, fill material test, and arranging specimen of soil.	Test pit, trench caisson method, Large diameter auger boring, Large diameter core boring.

Table 2.4 Method of Permeability Test

	Method of permeability test	Characteristics pumping in method
Pumping in method	Pond/canal Method	It is suitable for shallow and uniform ground, and easy to carry out
	Shaft method	It is suitable for shallow and uniform ground , and easy to carry out
	Hole bottom method	It is suitable for which bored hole has difficulty to stand
	Pucker method	It is suitable for hard rock or ground
Pumping out method	Equilibrium method	It is suitable for compressed or free ground water zone, with relative high permeability
	Non-equilibrium method	The method is classified into Theis method, Jacob method, recovery method, etc; depending upon the result arrangement will be arranged.

Table 2.5 Principle of Soil Test

	Test name	Values determined from test results	Use of test result
Physical properties test	Specific gravity of soil particle	Specific gravity of soil	Gradation analysis of soil by means of gravimeter, calculation of values of fundamental properties of soil
	Volume of water content	Ratio of water content	Calculation of values of fundamental properties of soil, rough judgment of soil properties (in case of natural water content)
	Liquid limit, plastic limit	Liquid limit, plastic limit Index of plasticity , index of consistency	Gradation analysis of soil by means of gravimeter , assessment of soil as material, judgment of cohesion stability in natural condition
	Grading of particle	Gradation analysis curve, effective diameter, coefficient of uniform, coefficient of curvature	Specification of soil as material, classification of soil
	Wet density	Wet density, dry density	Calculation of values of fundamental properties of soil, degrees of soil compaction
Dynamic test	Compaction	Ratio of water content- dry density curve Maximum dry density, optimum water content	Understanding of compaction characteristics, determination of construction conditions and design of embankment, calculation of compaction
	Searing	Shear strength constant (C,ϕ) C:cohesion ϕ: angle of internal friction	Stability analysis and calculation of earth pressure in relation to land sliding
	Consolidation	Ratio of porosity –Load curve, pre consolidation load coefficient of consolidation, coefficient of permeability, etc.	Calculation of volume of settling and settling velocity of cohesive soil
	Permeability	Coefficient of permeability	Judgment of acceptability as material, analysis of penetrating flow
Rock test	Specific gravity and percentage of water absorption for fine/ coarse aggregates	Specific gravity in saturated surface dry condition, Specific gravity of absolute dry condition, percentage of water absorption.	Judgment of lithology, calculation of value of fundamental properties
	Unconfined compression	Unconfined compression strength	Judgement of lithology

(2) Investigation for overall plan

1) Documents study

This is to collect available documents and comprehensively understand the condition of the whole project area. In order to promote subsequent investigation efficiently, the following documents should be collected as needed.

(a) Geological maps (1/50,000, 1/75,000, 1/200,000, etc.),
(b) Geological foundation maps (1/25,000, etc.),
(c) Soil maps (1/20,000, etc.),
(d) Soil investigation records,
(e) Construction work records and construction control records,
(f) Records relating to wells, ground water,
(g) Records relating to disasters.

2) Identification

This is the investigation to grasp the condition of the field of the intended area based on the obtained investagation data, and if necessary, sampling and sounding is made. This is the most important investigation for selecting the method of subsequent investigation. In large scale canal palnning, it is preferable to investigate such condition in cooperation with specialist in respective field, and it is particularly advisable for geologists to make broad and macroscope observations. In field reconnaissance, the following items should be investigated in particular.

(a) Outline of topography and geology,
(b) Geology and soil quality of outcrop,
(c) Unstable topography and disaster-striken districts,
(d) Sediments on surface layer,
(e) Slope situation,
(f) Land use, type and growth of plants,
(g) Situation of existing facilities,
(h) Location of spring water, water level of wells,
(i) Flammable gas,
(j) Testimonies of local elder citizens.

(3) Pre-feasibility study

1) Items of investigations

At this step of the investigations, the geology and soil properties of the proposed route are comprehensiverly clarified on the basis of the data obtained by documents study and identification. The investigation method is selected from the following items depending on the scale and importance of the canal.

(a) Sounding (penetration tests, etc.),

(b) Sampling by auger drilling,

(c) Sampling by machine drilling, and standard penetration test,

(d) Geo-physical underground exploration (seismic exploration echo-sounding, electric detection, geo-physical layer prospecting, etc.),

(e) Observation and sampling by test pits, trench, cutting, etc,

(f) Field water permeability test (pumping tests, injection tests),

(g) Observation and sampling by tunnel adits,

(h) Physical properties and mechanical tests of soil

2) Selection of investigation points

The investigation points are selected on the basis of the scale of canal, importance, presence of problems, topography and its items of investigation. The basic items are as follows:

(a) The standard interval of investigation along the canal route should be as suggested in Table 2.6.However, many points should be investigated where sounding or auger drilling is applicable,

(b) Investigations should be stressed at planned locations of important structures, such as pump stations, siphons, tunnel entrances and exits, supports of aqueducts, bridge foundation, wasteways, spillways, and gates, or in soft ground, permeable ground, expansive clayey ground, and fault areas.

Table 2.6 Standard interval for survey points

<table>
<tr><th>Works</th><th>Type of topography. Scale of works</th><th>Standard profile interval (m)</th><th>Remarks</th></tr>
<tr><td>Open canal (including covered drainage)</td><td>Wide alluvial plain
Narrow alluvial plain
Alluvia plain in valley
Flat alluvial upland
Undulating alluvial upland</td><td>300 ~ 600
200 ~ 400
100 ~ 200
300 ~ 600
150 ~ 300</td><td rowspan="4">Note
1) Survey interval should be longer if the ground condition is uniform.
2) Survey interval should be shorter if the ground represents changing topography.
3) Standard depth will be decided referring to Fig. 2.1.
4) Survey in traversing direction will be carried out as necessary.</td></tr>
<tr><td>Siphon</td><td>Long siphon
Short siphon</td><td>150 ~ 300
100 ~ 200</td></tr>
<tr><td>Tunnel</td><td>Short tunnel
Long tunnel</td><td>200 ~ 700
100 ~ 200</td></tr>
<tr><td>Structure</td><td></td><td>Decided as Necessary</td></tr>
</table>

(c) Since the results of this investigation are used not only as data for the design and construction, but also for the macroscopic understanding of the ground, it is preferable to undertake the investigation as deep as possible. For example, in reaching a solid layer considered to be the support layer of the soft ground, it is advisable to investigate deeper in order to confirm the thickness of this layer.

(4) Feasibility study

1) Items of investigation

After the determination of the canal route and the selection of the type of canal and facilities during the detailed design step, minute investigations are required for study of each object. The results are then used to determine the type and arrangement, and assess the design and construction of foundation treatments and construction cost. Although the contents of the investigation vary with the purpose, the investigation items, range, method and precision are determined depending on the kind and scale of each structure and ground condition. The standard investigation items and soil tests necessary for various parts of the canal structure are as shown in Table 2.7. The depth of investigation, which may vary depending on the type of structure, condition to be considered in relation to the design, and ground conditions, is generally as shown in Fig. 2.1 in a standard case.

2) Summarizing the investigation results

After the geological and soil investigations and tests, geological log chart or geological longitudinal section charts are plotted based on these results and geological and soil conditions of the final route are confirmed, while the findings are summarized as the fundamental data for the detailed design of the canal.

(a) Soil classification

A soil has its own physical properties, which are combined together to present complicated engineering properties. In the work dealing with soil materials, the design and construction are promoted on the basis of these engineering characteristics. Therefore, the classification of soil by engineering properties is technically of an immense value. The soil classification is applied in the following cases.

a) When predicting data on soil materials,
b) When determining the range of necessary field investigation for Pre-feasibility studies and Feasibility study,
c) When planning economic field investigation, test or indoor tests,
d) When performing supplementary investigations,

When the soil is classified, the suitability to the design is assessed with reference to the results of permeability, compaction, and shearing test, by referring to Table 2.8.

Table 2.7 Principal items of survey methods for foundation and soil material

Object	Principal items of survey	Method of survey	Soil test	Remarks
a. Foundation ground	Geological Structure, bearing	Collection, analysis of existing materials; boring and sounding		Investigation and tests for item a-c should be carried out in principle. Foundation ground means the ground which will become the foundation of canal, siphon and other structures.
b. Backfill material	Classification of soil, quantity of available soil, conditions of collection	Collection analysis and investigation of existing materials; boring, collection of samples	Volume of water content, specific gravity of soil particle liquid/ plastic limit, gradation	
c. Tunnel ground	Geological structure, geology (rock, rocky material) ground water	Collection analysis and investigation of existing materials; physical investigation boring	Unconfined compression Specific gravity volume of water absorption and others	
d. Backfill material	Characteristics of compaction, shear strength, permeability, erosion resistance, available quantity	Ground water survey, in-situ permeability, trial pit survey Trial pit survey, boring	Compaction, triaxial compression or unconfined shear, permeability and others	Survey and tests for item d-g will be carried out as necessary
e. Excavated ground of canal	Condition of ground water, shear strength, destiny, permeability	Ground water survey, in-situ permeability, collection of samples	Density, shear, physical properties test of soil and others	
f. Ground of canal	Ground water condition consolidation /settlement, shear strength, settlement, permeability	Ground water survey, in-situ permeability, collection of samples	Same as above	
g. Foundation for important structure, pump station, bridge, etc.	Groundwater condition, bearing, settlement, consolidation/settlement, shear strength	Ground water survey, in-situ permeability, collection of samples.	Same as above	
h. Foundation for Siphon	Ground water condition, bearing for execution, shear strength	Ground water survey, in-situ Permeability, plate bearing test foundation in-situ plate test		

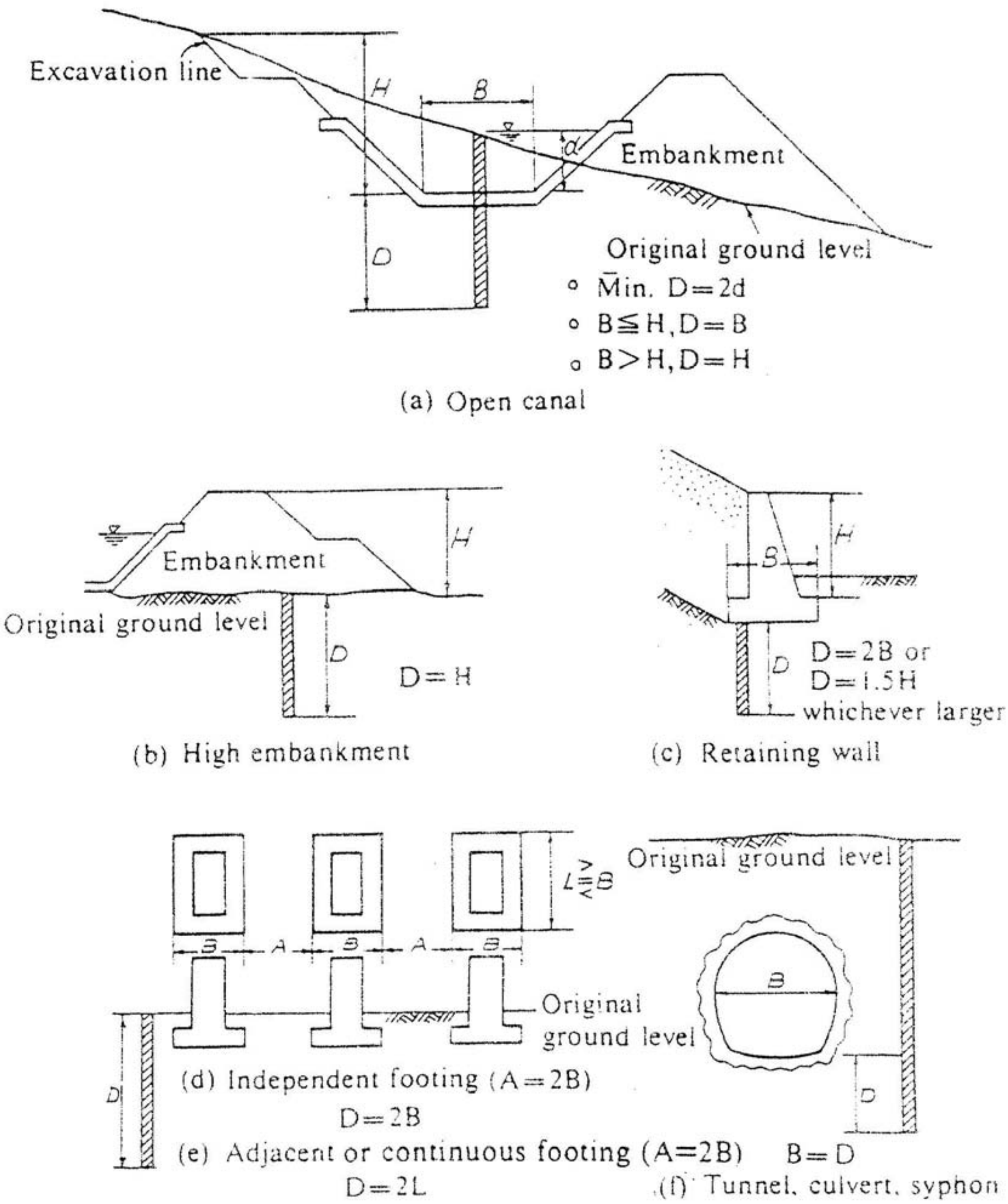


Fig. 2.1 Standard of test pit depth

Table 2.8 Engineering Characteristics of Soil

classification	Permeability Characteristics				Compaction characteristics				Shearing Characteristics				Erosion resistance		Expansion/Contraction		Frost action	
Symbol	Order	Permeability (while compacted)	Coefficient permeability (cm/s)	Necessity of lining	Order	Workability	Dry density (tf/m3)	Suitable machines	Order	Shearing strength	Cohesion	Internal friction angle	Order	Resistance	Order	Description	Order	Description
GW	14	Extremely large	$>10^{-2}$	Necessary	15	Very good	2.0-2.1	Pneumatic tire roller Vibration roller	1	Extremely Large	Small	Extremely Large	2	Extremely Large	14	Close to none	15	None - Extremely small
GP	16	Extremely large	$>10^{-2}$	Necessary	8	Good	1.8-2.0	Pneumatic tire roller Vibration roller	3	Large	Small	Large	3	Extremely Large	16	Close to noe	17	None -Extremely small
GM	12	Medium -Small	10^{-3} - 10^{-6}	Necessary - Not necessary	12	Good	1.9-2.1	Pneumatic tire roller Tamping roller, Vibration roller	7	Large	Average	Large	5	Large	12	Extremely small	10	Small -Average
GC	6	Small	10^{-6} - 10^{-8}	Not necessary	11	Good	1.8-2.1	Pneumatic tire roller Tamping roller	9	Large	Averate	Average	4	Large	10	Small	11	Small -Average
GW-GC	8	Small	10^{-5} - 10^{-7}	Not necessary	16	Good	1.8-2.1	Pneumatic tire roller Tamping roller	4	Large	Average	Large	1	Extremely Large	11	Small	12	Small -Average
SW	13	Large	$>10^{-3}$	Necessary	13	Very good	1.8-2.1	Vibration roller	2	Extremely Large	Small	Extremely Large	8	Average	13	Close to none	14	None -Extremely small
SP	15	Extremely Large	$>10^{-3}$	Necessary	7	Normal	1.6-1.9	Vibration roller	6	Large	Small	Large	9	Average	15	Close to none	16	None -Extremely small
SM	11	Medium -Small	10^{-3} - 10^{-6}	Necessary - Not necessary	10	Normal	1.8-2.0	Pneumatic tire roller Tamping roller, Vibration roller	8	Large	Small	Large	10	Small	9	Extremely small - Average	7	Small -Large
SC	5	Small	10^{-6} - 10^{-8}	Not necessary	9	Normal	1.7-2.0	Pneumatic tire roller Tamping roller	10	Avarage	Average	Average	7	Average	7	Small - Average	8	Small - Large
SW-SC	7	Small	10^{-5} - 10^{-8}	Not necessary	14	Good	1.7-2.1	Pneumatic tire roller Tamping roller	5	Large	Average	Large	6	Large	8	Small - Average	9	Small -Large
ML	10	Medium -Small	10^{-3} - 10^{-6}	Necessary - Not necessary	5	Normal	1.5-1.9	Pneumatic tire roller Tamping roller	12	Average	Small	Average	-	Extremely Large	6	Small - Average	1	Average - Extremely Large
CL	3	Small	10^{-5} - 10^{-8}	Not necessary	6	Normal	1.5-1.9	Pneumatic tire roller Tamping roller	11	Average	Large	Small	11	Small	5	Average	3	Average - Large
OL	4	Medium -Small	10^{-4} - 10^{-6}	Not necessary	3	No good	1.3-1.6	Pneumatic tire roller Tamping roller	15	Small	Unknown	Unknown	-	Extremely small	4	Average - Large	4	Average - Large
MH	9	Medium -Small	10^{-4} - 10^{-6}	Not necessary	2	No good	1.1-1.5	Tamping roller	14	Average - small	Large	Small	-	Extremely small	3	Large	2	Average - Extremely Large
CH	1	Small	10^{-6} - 10^{-8}	Not necessary	4	No good	1.2-1.7	Tamping roller	13	Average - small	Large	Small	12	Samll	2	Large	5	Average
OH	2	Small	10^{-6} - 10^{-8}	Not necessary	1	No good	1.0-1.6	Tamping roller	16	Small	Unknown	Unknown	-	Extremely small	1	Large	6	Average
Pt	-		-	-	-		-	-	-	Small	-	-	-	Extremely small	-	Extremely Large	13	Small

Notes: 1. GM: Among GF, the one which fine-grained fraction is mainly silt. GC: Among GF, the one which fine-grained fraction is mainly clay. SM: Among SF, the one which fine-grained fraction is mainly silt. SC: Among SF, the one which fine-grained fraction is mainly clay.

2. Rank order is a conceptual order in each property.

(b) Geological log chart and longitudinal sectional charts

The results of field investigations such as of drilling and sounding are summarized and geological longitudinal sectional charts are plotted.

2.2.3 Meteorological and Hydrological Investigations

Meteorological and hydrological investigation may be carried out including the collection of data and observations on the precipitation, river water levels, discharges, the condition of river courses and other conditions in the project area.

(1) General

Items to be investigated in meteorology and hydrology are air temperature, rainfall, evaporation, water level and discharge of rivers, ground water level, and characteristics of the river basin. These data are used for designing, construction and maintenance of canals such as the determination of the design discharge and scale of canal, design of related structures and establishment of constructions planning. Generally, these investigations are carried out continuously from the Identification stage to the Pre-feasibility study stage. The findings are used for the determination of basic conditions for making the Pre-feasibility study and the Feasibility study. In the stage of the investigation for feasibility, these findings are checked again for the review of the hydraulic design of facilities and structural design and for the preparation of the construction plan and the operation and maintenance plan. It is also necessary to collect the latest information on every investigation item because a considerable period is consumed from planning to construction. Particulars relating to the drainage plan, the runoff ratio and other factors must be sufficiently investigated because they are subject to change due to topographic and ground surface conditions. Therefore, from the planning stage, measuring instruments for rainfall, river discharge and other data should be installed at necessary places in order to obtain continuous measurements. Observation should be continued at such places in the operation and maintenance stage to provide a basis for management decision.

(2) Meteorology

Since open channels are usually located on plains, the data of existing observation station in adjacent areas can often be effectively used. Generally, in a large–scale project planning the study area extends over a very vast range. The meteorological data of all stations in and around the area should be collected for over 10 years if possible to grasp the meteorological conditions of the area and to put it to good use for planning and designing. The rainfall observation station should be not being located in a place exposed to wind or flooding. The instrument may be either an automatic or an ordinary rain gauge. If an ordinary rain gauge is used, measurement must be made at a specific time every day. The following observed results need to be summarised in fixed forms.

1) Hydrological data relating to irrigation canal planning
   Mean rainfall for rainy and dry seasons, mean annual rainfall, mean monthly temperature, mean monthly rain days, maximum successive no-rain days, most frequent wind direction, maximum wind velocity, etc.
2) Meteorological data relating to drainage canal planning
   In addition to the items in 1), maximum daily rainfall, maximum hourly rainfall, etc.

(3) Hydrology

1) Water levels and discharge of river

The water levels and discharges of rivers necessary for planning, design, construction and operation and maintenance of canal are explained herein. For the irrigation canal planning, it is preferable to observe water levels and discharges over an irrigation period at the intake site. For drainage canal planning, since the high water level of the main drain and its duration are important, it is necessary to collect data for a longer period. In particular, if the main drain is a natural river, the water level and discharge may be affected by the river improvements condition, changes in river bed, soil and water conservation measures of upstream and other changes in ground surface. Therefore, it is desirable to install new instrument and to observe the water level and discharge when necessary. Principal items for observation are as follows;

(a) Flood level, discharge
(b) 95-day water level, discharge (if possible)
(c) Mean annual water level, discharge (if possible)
(d) Ordinary water level, discharge (probable water level and discharge occurring more than 185 days in a year) (if possible)
(e) Low water level, discharge (probable water level and discharge occurring more than 275 days in a year) (if possible)

(f) Base flow water level, discharge (probable water level and discharge occurring more than 355 days in a year)

2) River condition

Investigation of river condition is made to understand the circumstances of river to which canals are connected in order to maintain the functions of the canals. Investigations are done mainly about the river bed shape (plain, longitudinal section), river bed fluctuation and river bed materials. In determining the sills of intake and drainage outlets and in designing the head race and wasteway, river bed fluctuations must be investigated. If past data are not available, fluctuations in the river bed must be investigated by interviews or other means.

3) Ground water level, etc.

Since ground water, spring water and leakage water from facilities are influential in determining the canal structure design and construction method, they must be carefully investigated from the planning stage. Aside from the findings obtained through review of existing data and field reconnaissance, detailed investigations should be made at the time of drilling and other geological investigations, in order to clarify the ground water level (final stabilized water level), locations and volume of spring water and leakage water in and around the study area.

(4) Others

1) Water quality

The function of a canal may be restricted and its useful life may be shortened by deposits of suspended load and sand on the canal walls and bed. To avoid such trouble, the water quality of river and drainage water must be investigated. That is, sediment load, suspended solid, pH and salt concentration of water must be analysed.

2) Drift

Drift causes stagnant flow, head loss, closing of conveyance capacity of canal and other troubles relating to the safety and maintenance of canals. It is hence important to investigate sufficiently the volume and kind of drift likely to be inducted into the canal.

3) Others

Disaster records, various data on ground surface, water temperature and other conditions need to be investigated as required.

2.2.4 Investigations of Site Conditions

In the investigation of site conditions, social conditions of canal system, construction condition, environmental condition and other conditions relating to land acquisition and real rights must be known through data collection and reconnaissance.

(1) General

For the construction of canals, since social conditions and environmental conditions other than natural conditions are very important, these conditions are investigated as required in parallel with or ahead of other investigations.

(2) Investigation on social conditions

Social conditions are investigated to collect data for land use, improvement of farm management, and protection and coordination of the existing facilities and rights.

1) Land use and regional development program

Canal layout is an important factor to define the future land use and living environmental conditions of the surrounding area by providing irrigation or a drainage canal. Therefore, present land use housing condition, farm management status and regional development program must be sufficiently investigated in order to consider the land use in the local society in harmony with the living environment.

2) Irrigation and drainage systems

Understanding of existing irrigation and drainage systems is important in determining the route and scale of canals. Sufficient surveys of existing systems must be made from the planning stage, where consideration is being given to the maintenance and improvement of functions of existing facilities.

3) River structures and water balance

The position of river structures in associated irrigation and drainage systems, their relation to water conservation and water use, existing irrigation site must be investigated. Since these items are intricately related to each other, investigations must be made from the viewpoint of the entire associated irrigation and drainage systems.

4) Others

For the canal route selection, existing structures mining rights and other rights in the area must be investigated. Land slide and erosion control, various regional designations, buried cultural properties and other relevant items must be investigated at the same time. It must be noted also that there may be limitation imposed by by-laws and regulations. Since the routes of canals are not always determined by the technical and economical conditions, social factors must be considered from all angles.

(3) Investigation of construction conditions

In investigation and design of canals, the location and scale of construction facilities, procurement and handling of construction materials, availability of power and other conditions must be investigated.

1) Facilities for construction

Temporary facilities for the construction of such items as access roads, borrow-pits, and spoil-banks are the basic and essential requirements in canal construction. Therefore, preliminary investigations must be done when planning the canal routes so that significant changes may not be required in the construction stage. In the case of a long tunnel, in particular, since the route is closely related to the construction plan for adits, inclined adits, and shafts for use in construction, the most economical overall construction method must be considered. Access roads must be investigated in relation to location, trafficability and other field conditions. It is important to plan the road in consideration of other social conditions, not merely for economy of construction work, as the road may be related to existing or future public roads or roads for operation and management.

2) Construction equipment and materials

Since the canal and related structures are constructed in successive steps, the balance of supply and demand for construction materials and equipment must be checked to ensure if they can be produced according to the progress of the construction work. If special materials or equipment are required, then their availability must be confirmed in advance.

3) Others

The construction of canal depends considerably on the labor force. The investigation into availability of labor may be sometimes necessary. The supply of electric power must be confirmed if a large structure is to be constructed in the canal system. In other investigation, to select the best construction method, the ground water level and foundation condition must be studied and the method of disposing materials wasted in the field must be also studied.

(4) Environmental investigation

Appropriate consideration must be given to the preservation of living and natural environments in relation to the construction of canals. Therefore, suitable investigation must be made with due regard to the laws and regulations concerned.

1) Investigation on living environments

In the selection of the canal route, the location of community and public infrastructures will be suitably investigated in order to protect the living environments and also to make use of the disaster preventive functions of canals. It is necessary to find the most appropriate design and construction method with due consideration to vibration, noise, traffic disturbance and other nuisances of the construction work. Special attention must be paid to the selection of the construction method and safety measures in the construction work particularly near urban area.

2) Investigation on natural environments

If there are any risks of water pollution, interference with ground water or of land subsidence, foundation conditions, ground water level and the condition of underground streams will be investigated and proper countermeasures will be taken to prevent such phenomena.

2.2.5 Investigation on Water Management and Operation and Maintenance of the Canal System

The particulars required for the study of water management and operation and maintenance of the canal system must be decided through data collection and inquiry in the project area. The canal system, water management system and operation and maintenance (O&M) facilities must be planned in parallel with study of the O&M organization to be established after the completion of the facilities. Therefore, in the investigation relating to O&M, the system and organization of O&M on the existing similar projects are firstly studied to obtain information related to the methods and systems used for the water management and O&M organization. By reference to these data, the water management method and level are determined. In case of assignment of the management, the scope of the assigned works, allocation of assignment expenses and method of operation and maintenance will be studied.

3. DESIGN CONCEPTS

3.1 General

3.1.1 General Concepts

Canals must be designed for their required function, economy and safety, proper water management. To fulfill these functions various appurtenant facilities of the canal system must be designed effectively to complement each other.

A canal system is a combination of various appurtenant facilities to enable it to fulfill its functions. For this purpose, it is necessary to realize an appropriate design of the canal system. In this manual, the design of a canal system is called “overall design”, and that of individual facility is called “canal or structure design”.

(1) Overall design of irrigation canal

The designed level at a starting point of an irrigation canal is determined by taking into consideration common requirements as well as the location and type of the intake structure. Although that of benefited area is determined by taking into consideration of the elevation and location of farm land at the tail-end, irrigation method and the designed level to the terminal canal. Consequently, the plan of water requirement, designed level and abutment of a canal is determined. The water surface line and its gradient in the irrigation canal can be estimated by connecting the designed level at the beginning point with that of expected lateral canals. In the overall design, the alignment of the canal route and location of appurtenant structures are determine, taking into consideration of the stability and safety of a canal, running cost and environmental conditions.

(2) Overall design of drainage canal

The discharge and designed level at the end of a drainage canal are first determined in the overall design of the drainage canal, considering areas that would benefit from the drainage development. The present drainage system and present land use under two drainage conditions i.e. flood and normal condition. The drainage method (gravity, pumping or combined), location of the junction of drainage canals with river/sea and the designed level at the junction are determined by taking into consideration the design and water level of the river/sea at the junction. It is important that the overall design of the drainage canal, the location of drainage facilities and alignment of drainage should be determined under these two drainages.

(3) Main items to be considered in overall design

1) Conveyance capacity

(a) The conveyance capacity of canals is mainly controlled by the structure; an appropriate capacity for each structure is required to provide the conveyance capacity of a canal system. In the overall design, various discharges of the canal should be examined for each facility.

(b) In irrigation canals, water use conditions of main canals and terminal canals are not similar according to the cropping patterns. The terminal canals supply water hourly to the field. While in main canals, the designed water use is usually given continuously by daily unit. Conveyance capacity of main and terminal canals must be designed by taking into account the required design conditions and hydraulic conditions.

(c) Some canal systems are compounded by pipeline structure with quick response of discharge change. Open channels with slow response of discharge change and pumps. In the viewpoints of continuous water conveyance facilities having buffer effects i.e. regulating reservoir, are often required at the functions. When an open channel is employed as a conveyance canal, it is desirable to introduce a regulating reservoir for adjusting time lags and effective water use. In the case that pumping stations are provided on the open channel, it is sometimes required to provide a regulating reservoir to enable to continuous pump operation. For drainage purpose, it is also required to provide retarding basin (reservoir which stores flood temporarily) for continuous operation of plural pumps.

2) Functions of regulation and distribution of the irrigation canal

In the irrigation canal, discharge is not fixed throughout the year. The design discharge of irrigation facilities is generally determined by the maximum seasonal requirement., However, It is necessary to check that the facilities can maintain the designed level at turnouts and regulating function even when the discharge in the canal is less than the design discharge.

3) Functions of regulation and stability of the drainage canal

It is possible that the discharge in excess of the designed one occurs in the cases of the drainage canal, functions of stability and regulation of the drainage canal must be checked in such cases.

4) Water management and maintenance facilities

It is required to provide complete water management and maintenance of facilities proportionally as the scale of a canal system becomes larger. Therefore, it is necessary to study the layout and structure of management facilities in order to carry out water management and maintenance works economically and smoothly.

5) Safety of facilities

(a) In irrigation canal, such occurrences like breakage of facilities or sudden mechanicals stoppage of gates and pumps due to floods, landslide and others problems may happen. In order to prevent damage due to such breakage or stoppage, it is required to provide adequate wasteways, spillway and regulating reservoir in the irrigation system.

(b) When the function of an irrigation canal system is not well balanced, the irrigation canal system is controlled by the facility with the lowest capacity. Therefore, the capacity and durability period of individual facilities must be determined by taking into consideration the entire function and capacity of such facilities.

6) Harmony with environment

Canal routes, and type and scale of facilities must be determined mainly to meet the objective of the canal system. In addition, harmony with the environment and local conditions must be considered in the concept of the project. At the same time, the overall design should be well marched with the related regional development projects.

(4) Canal or structure design

The canal or structure design must be conceived to provide the required function and safety to hydraulic and structural conditions as well as required functions at the stage of the overall design.

3.1.2 Procedure of Design

The design of canal must be made based on a well-established working plan and through adequate procedures, according to a comprehensive judgment on the planning conditions of irrigation and drainage, natural and socio-economic conditions and other complex problems related to the project areas. The design of the canal should be based on a well-established working plan and through adequate procedures, after effective studies on complicatedly related conditions such as economical, safety and working conditions as well as natural and socio –economic conditions. The procedures of the overall design are shown in Fig. 3.1. And the procedures of the canal and structure design are shown in Fig. 3.2. These figures indicate the related studies for the overall design and canal/structure design respectively as flow charts, but some studies may be omitted according to the scale of the system or facility. Therefore, the design works require a well coordinated plan for the related contents of each study. The overall design includes confirming the design discharge and designed level, studying canal types, canal routes, types of facilities, head allotment and so on. Since these studies are related with other studies, each study should be well adjusted for the adequateness of the whole system of canal, and in this respect, it is important to repeat studies for satisfactory results.

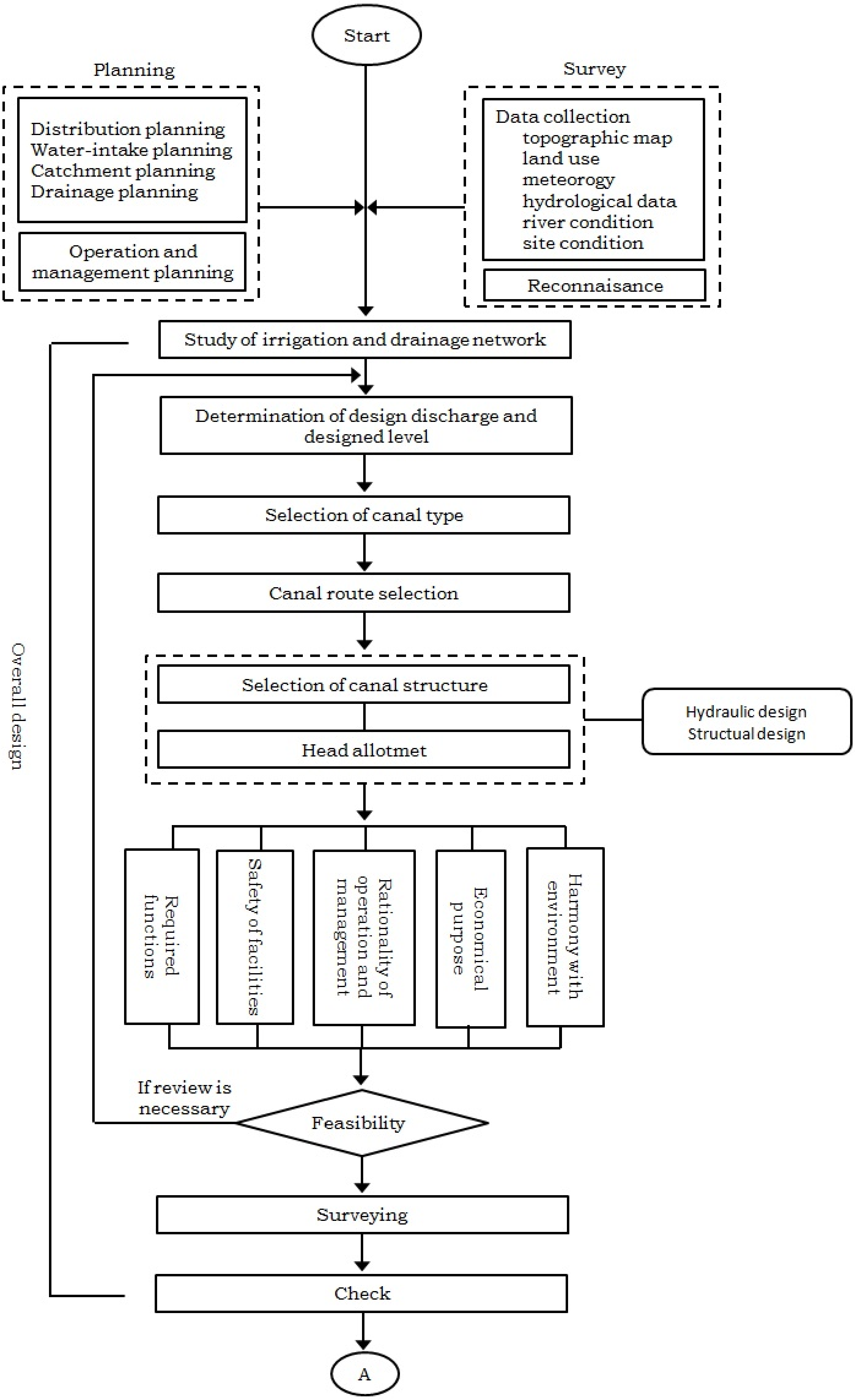


Fig. 3.1 Procedure of the overall design

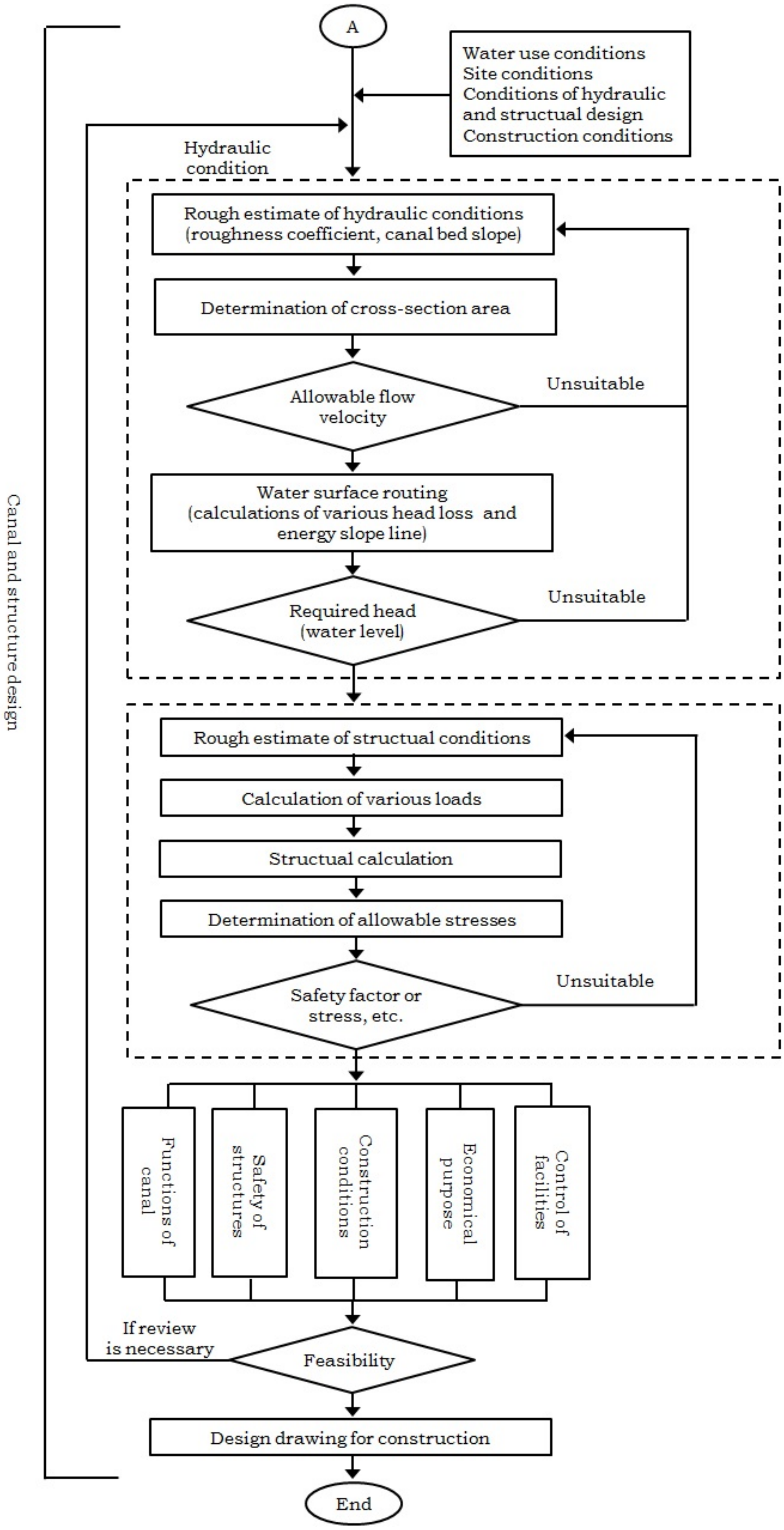


Fig. 3.2 Procedure of the canal and structure design

The canal and structure design require efficient works under the procedures of working plans to determine the facilities which satisfies the proposed functions, safety, economical and technical conditions for the project execution, also based on the conditions given to the canal system. Therefore, efficient studies are required to make design well adjusted by using examples, data and records, monographs and standardized designs. Since these design data are very effective to other similar projects at the planning, execution and management stage, it is important to file and preserve these data.

3.1.3 Design Discharge and Designed Level

Prior to the canal design, the proper design discharge and designed level of each facility must be determined by confirming their basic requirements, such as planned canal discharge of irrigation and drainage, planned design level and layout planning of irrigation and drainage systems.

(1) Design discharge

In this manual, the design discharge is the planned maximum discharge of the irrigation plan of seasonal and irrigation blocks, and the planned maximum discharge of the drainage plan. And the minimum discharge is the discharge of the planned minimum discharge and the most-frequent design discharge is the discharge which most frequently occurs in the plan. Although the capacity of canal and structures is generally controlled by the maximum discharge, it must be checked against the discharge of less than the maximum discharge for the determination of the canal section and structures.

1) Irrigation canal

The design discharge of an irrigation canal is the planned maximum seasonal discharge or maximum discharge at each irrigation block of the canal. When such cases are supposed that planned water is not diverted at the upstream portion of the canal or flood discharge cannot be avoided to flow into the canal, these conditions must be also considered to determine the design discharge. Generally, the design discharge is determined by taking consideration related diversion facilities, wasteway, spillways, in the block of a canal system. Therefore, the design discharge is determined after alternative studies of total construction costs of each block of the canal including construction costs of wasteway, spillways and related river improvement are examined.

2) Drainage canal

In general, the design discharge of drainage canals is the maximum drainage discharge of the drainage planning. In the irrigation planning, the discharge that is over the designed discharge is not supposed to flow in the canal, but in the drainage planning, it can be supposed that the discharge, which is over the planned discharge make trouble in drainage canals. The drainage area of a drainage project rarely changes but the drainage discharge may change due to changes in the hydrological condition, land use and environmental conditions. Therefore, it is necessary to promote the project by checking and taking necessary counter-measures regarding these conditions at each of the planning, design and construction stage.

(2) Designed level

In this manual, the designed level is the water level, which can flow the planned seasonal maximum discharge at each irrigation block in the irrigation planning, and the planned maximum discharge in the drainage planning.

The designed level is an important value to determine the scale, section and function of facilities. It must be checked and determined whether such facilities can function safely and satisfactorily.

1) Irrigation Canal

The highest designed level is, as mentioned above the seasonal highest water level or the highest water level at each irrigation block of the canal. However, when such a case is supposed that planned water is not diverted at the upper stream portion of a canal, the designed level is determined by taking into consideration the addition of the discharge corresponding to the volume above mentioned. The water level at the minimum discharge is called the lowest design water level and the water level, which occurs most frequently, is called the most-frequent design water level. The lowest design water level and the most-frequent design water level are used to check if turnouts, drops, and measuring and regulating facilities function smoothly. The highest design water level at an arbitrary point of the irrigation canal can be estimated by interpolating two water levels at the beginning and end, in case the canal system is simple. In case the canal system is complicated, such water level cannot be easily estimated due to the topographic condition and provision of various kinds of facilities. In the overall design, such water level must be determined, taking into consideration the scale and type of facilities and canal routes.

2) Drainage canal

The highest drainage water level is the highest water level, which is used in drainage planning. The water level at the minimum discharge of a drainage canal is called the lowest drainage water level. The water level in the drainage canal must be determined so as not to exceed the elevation of the outlet of under drains for draining groundwater. The designed level in the unlined drainage canal is required to be determined not to exceed the ground elevation, if damage due to seepage from the canal is anticipated. The designed level, canal section and scale of the drainage facilities must be determined so that the drainage canal can drain the designed drainage discharge safely, taking into consideration ground elevations at each section of the drainage canal. The water surface in the canal is determined by hydraulic backwater calculation on the ordinary discharge and for river revetment planning. Based on the results, drainage facilities such as covered drains and river revetment must be designed. Although the drainage method (gravity or pumping) has been usually determined in the drainage plan, it may change due to changes of land use and social environmental conditions after the determination of these factors. It is, therefore, necessary to promote the project by checking these conditions at each of the planning, design and construction stage.

3.1.4 Selection of Canal Type

The canal type must be determined on the basis of the designed discharge and designed level in consideration of the natural and social environments of the route, economy, water use, water requirement, operation and maintenance and other conditions, so that the purpose and function of the entire canal system may be fully achieved. The selection of the canal type greatly affects the function of the entire canal system, and significantly affects the construction costs of canals. It is, therefore, necessary to consider the conditions of the costs and future water management and maintenance system in the selection of the canal type, aiming at the entire fulfillment of its purpose and function. Types of canal are open channel type including mainly open channel, tunnel and siphon, and pipeline type, mainly consisting of pipelines, and compound type of open channels and pipelines. The advantage and disadvantage of these types are as follows:

(1) Open channel type

1) Since the construction cost of an open channel type is generally lower than that of a pipeline, the open channel type is widely employed. When due to topographic conditions, complicated series of structures such as open channels, tunnels, siphons, etc. are required or an open channel has to be larger than a pipeline, the construction costs of an open channel type may be higher than that of a pipeline type.

2) When the discharge of an open channel type is regulated by gate operation, the transmission speed of the discharge change varies with such factors as reflection by structures, variation of cross section of canals and others. The transmission speed in the open channel type is slower than that of pipeline type. Consequently, it takes a considerable time to change the flow into a stable condition in an open channel type.

3) Since an open channel type generally more flexibility in capacity than the pipeline type has, an open channel type can be used for drainage as well as for supply as a dual-purpose canal.

4) Floating materials in an open channel type, which may limit its conveyance capacity, can be easily removed. Nevertheless, west materials such as trash and garbage are often throw into canals. Countermeasures against such acts are required.

5) Safety facilities to protect the general public and operation and maintenance personnel are required for open channels. Countermeasures against these problems were specially required.

(2) Pipeline type

1) The alignment of a pipeline can be designed regardless of topographic conditions providing the pipeline is always set below its energy line.

2) When the discharge in a pipeline type is regulated by operating gates or valves, the discharge change is transmitted as a water hammer wave and the transmission speed is relatively higher. This speed of closed type pipelines is higher than that of open type pipelines. It is, therefore, possible to change the discharge quickly as required. But careful operations are required not to grow an excessive water hammer pressure.

3) Since the losses of water in a pipeline type are less than in an open channel type, an effective use of water resources can be attained.

4) Irrigation water contains floating materials such as dust, silt and fine sand. Deposits of such materials in a pipeline cause reduction of its conveyance capacity. Since it is difficult to remove such materials from a

pipeline by maintenance works, it is necessary to provide settling basin trash-removing facilities at the inlet to the pipeline in order to prevent the intrusion of floating materials. In general, the pipeline type canal is not suitable for drainage purposes since as inlet to the pipeline could be choked by such materials.

5) A pipeline type is a facility to convey discharge using the difference of energy at two points. Since the pipeline type has less flexibility in capacity than an open channel, it is not used as drainage canal or dual-purpose canal.

6) Since the pipeline type is buried in the ground, more effective use of land resources is possible.

7) Although a pipeline is generally costlier than an open channel in construction, the pipeline type is widely employed for small-scale irrigation facilities mainly due to easier operation.

The selection of the canal type must be made taking into consideration its advantages and disadvantages as described above. In recent years, the economic advantage of an open channel type has gradually declined due to the increase in construction costs of related facilities resulting from topographic limitations. The pipeline type has been widely adopted even in large-scale irrigation schemes.

(3) Compound type of open channels and pipelines

1) Due to difficulty of land acquisition for open channels in urbanized area, preservation of water quality and labor saving of water management pipeline type is planned. Such cases have prevailed in that of the upstream portion of a canal is an open channel type and the downstream portion a pipeline type or vice versa and in a compound type of an open channel type and pipeline type.

2) In compounded type canal, satisfying the continuity of the water flow at the junction point of two types is required taking into consideration the characteristics of the two types and in the determination of regulating capacities of facilities or alignment of spillways, etc. In an open channel type canal, check gates can control the water level of the upstream portion; therefore, in water management having an upstream priority, this can be attained by operating diversion gates. And a certain time is required to induce a change of the discharge. While in a pipeline type canal, a change of the discharge at the inlet a pipeline can be made by opening a valve at the

end point of canal. Moreover, an extreme short time is required to change the flow condition in a pipeline type canal compared with an open type canal.
3) A compounded type canal, possessing the merits and demerits of an open channel type and a pipeline type canal, requires satisfactory overall design by taking into consideration its operation for water management after construction.

3.1.5 Canal Route Selection

The canal route must be selected based on the design discharge and designed level by considering the entire canal alignment, the purpose of canal, safety and economy of the structures in addition to the natural and social conditions existing along the route.

(1) General conception for the canal route selection
The canal route must be selected so that the canal can maintain the required design discharge and designed level, taking into consideration the type, scale and location of the canal and its structures. Problems related to land acquisition and the change of conventional water rights may occur in the selection of the canal route. Therefore, the most effective canal route must be selected carefully on a map through alternative studies before pegs are actually driven in the field.

(2) General condition for the canal route selection
1) Canal system
(a) The canal route must be selected so that the canal can irrigate by gravity as far as possible within the range of the available water head. If the project area includes some elevated land, then the canal route must be selected, by considering the use of pumped irrigation to such areas, and by comparing the construction costs and future water management and operation costs between these two irrigation methods.
(b) A drainage canal is generally aligned at the lowest point in the project area to facilitate drainage by gravity. If gravity drainage cannot be applied to the entire project area for topographic reasons, then the introduction of the pumped drainage should be alternatively studied at the stage of the canal route selection. The final canal route must be selected by a comparative study. If pumped drainage is applied to the whole project area, then

provision of plural pumping stations at appropriate places must be considered to minimize running costs of pumps.

(c) Locations of spillways, wasteways and turnouts must be determined from map studies of field conditions of benefited area and from surveys of target rivers in relation to the project area.

(d) If the irrigation canal is long and its purpose is to irrigate upland field mainly, then the necessary and possibility of establishing regulating reservoirs must be considered at the stage of the canal route selection. There are, in general, several advantages by providing regulating reservoirs such as decrease in construction costs due to the reduction of required canal section, simplification of canal structure, etc.

(e) An irrigation canal is closely related to environmental conditions and may have other roles in the social conditions of the region. This relationship must be considered in the design of rehabilitation, improvement works for existing irrigation facilities, Regional development plans or projects must be studied thoroughly, and the results must be considered in the canal route selections.

2) Others

(a) The canal route of a pipeline must be straight and short as much as possible. The route of an open channel must be selected to avoid high-banking or deep-cut section, taking into consideration the available water head and allowable velocity.

(b) Soil conditions and interference with houses and public traffic due to the construction of canals must also be considered.

(c) It is advisable for the safety of the canal as well as for public safety to avoid as much as possible the setting of the canal route on banking.

(d) In some project area where considerable water head is available from the intake structure to the benefited area, this head can effectively be used for hydroelectric power generation to decrease operation and maintenance costs of the facilities. Such possibility for the effective use of resources can considered at the stage of the canal route selection if possible.

(e) If the irrigation canal is shared by non-irrigation projects, then social and economic damages may be occurred by the suspension of non-irrigation water supply during maintenance of the non-irrigation facilities. It is, therefore, necessary to consider the use of a parallel canal or double-section canal.

(3) Procedure for canal route selection

Survey and investigation necessary for the canal route selection are described in Chapter 2.2. Here is described the procedure for the canal route selection based on surveys and investigations. It involves two steps i.e. the canal route finalization to be made at the investigation and planning stage, and then the canal route finalization to be made at the detailed design and construction stage. The canal route proposed is to select the most appropriate route on available topographic maps after a comparative field study of several canal routes with regard to purposes and functions of the canal, costs and safety of the facilities. The canal route finalization is to finalize the canal route and to realize drawings of the plan based on the proposed route and results of the detailed investigations in the field and to drive pegs at the centerline of the route in the field details of each step are as follows:

1) Planning of proposed route

(a) The proposed route is generally made on a topographic map at a scale of 1/2,500 - 1/5,000 covering the entire project area. The scale of the proposed route may be made directly by the field surveys.

(b) Basic designs of several canal routes must be made and compared, and the most appropriate design adopted considering the alignment of structures, head allotment, effectiveness for planning, costs, etc. Field survey works must be carried out to collect all the data necessary for the comparative study such as data for design, construction and operation of the facilities, social condition in addition to topographic and geological data.

2) Canal route finalization

(a) Strip topographic maps at a scale of 1/500-1/1,000 must be prepared to show the most appropriate canal route and topography of both side areas of the route in a width of 30-100m. These maps must be highly accurate since the maps will be used for the canal route finalization, design and construction.

(b) The most appropriate canal route as shown on the strip topographic maps must be finalized, taking into consideration types, scales, water level and sections of canals and structures.

(c) The finalized canal route must be checked in the field, and additional soil mechanical and geological investigations must be made as necessary. Then pegs must be driven at the centerline of the route in the field. If unsettled problems remain, then such problems should be settled before driving pegs since they might induce a change in the finalized route.

3.1.6 Selection of Canal Structures for Open Channel Type

The water conveying facilities in the open channel type, such as open channel, tunnels, culverts and siphons must be selected to ensure the purpose and function of the entire canal system in consideration of safety and economy of the structures, the topography along the route, natural and social conditions such as the land use conditions along the route.
A canal system consists of water conveying facilities such as open channel, tunnels, culverts, aqueducts, siphons, drops, chutes, and other facilities such as turnouts, water-measuring devices, regulating facilities, safety duties, operation and maintenance facilities, appurtenant facilities. The selection of the most appropriate water conveying facilities must take into account the canal type and route, to maintain all functions of the entire canal. Since water conveying facilities occupy the main portion of a canal system and could affect to a large measure the function and costs of the whole canal system, adequate studies must be made on the adequateness in the selection of the stability and costs of the structures. Turnout works, water measuring devices and regulating facilities, etc. are closely related to water conveyance facilities in their function. Therefore, the selection of the water conveying facilities must be made, taking into consideration the location, type and scale of these structures. Since the pipeline is a monolithic water conveying facility, the selection of the water conveying facilities is not required. The type and arrangement of the appurtenant facilities must be, however, considered.

(1) Open channel

1) An open channel may be classified into three types i.e. retaining wall type, lined and unlined. The type of the open channel must be selected by comparative study with regard to the objective, stability, social, conditions, construction costs, and operation and maintenance of the canals.
2) In general, an open channel is hydraulically advantageous compared with a pipeline type canal. It may be economical when cutting and banking are well-balanced in earth work volumes.
3) Excessive cutting and banking must be avoided as far as possible for stability as well as to minimize costs of the canal. In case that such cutting and banking are made in an open channel when they are in short length and preferable to siphons of culverts. Foundation conditions and stability must be fully considered in the design of such canals.

4) The unlined canal is generally adopted as drainage canal, which does not require protective measures against water leakage, erosion and source at curve and confluence. When erosion and source are anticipated due to steep topographic conditions, provision of drops or chutes in a gentle slope of an unlined canal may be preferable. At confluences and curves where erosion and scour may be anticipated, lining or revetment may be required.

5) A lined canal is a canal where stable condition of a canal are secured by gentle slope of the banking material itself, and the slope surface is covered by relatively thin material to prevent water leakage and to render its surface smooth. A retaining wall type canal is one in which side walls support internal and external water pressures and external earth pressure. Both lined canal and retaining wall type canal are used to prevent water leakage and to reduce the cross-section area of the canal. When there are no restrictions in respect of land acquisition and other topographic requirements, the lined canal is more advantageous in terms of cost than the retaining wall type canal. This advantage becomes greater as the scale of the canal system becomes larger. The followings are conditions for the adoption of the retaining wall type canal:

(a) Where the cross-section area of the retaining wall type canal is smaller and its construction costs, including land acquisition cost is lower compared with the lined type canal.

(b) Where there are no foundation treatment problems for a retaining wall type canal and its construction cost is lower especially for each works.

(c) Where construction of the lined canal includes considerable amount of earth and rock work and costs due to topographic and geological conditions where a canal is aligned on the steep slope side on through a high cutting saddle-backed condition.

(d) Where an open channel is provided for a short section between a tunnel and siphon, and where the retaining wall type canal is lower or equal to the total construction cost of the lined open channel and transitions.

(2) Tunnel

A tunnel is employed where construction of an open channel may be quite difficult due to topographic conditions where a canal route must be aligned on the higher portion in mountains or terraces and where the total construction cost of the tunnel is lower than that of an open channel due to shorter length of the tunnel. The followings are considered in the adoption of a tunnel:

1) Since the construction costs of a tunnel is two to three times higher than that of an open channel, and the cost may increase further when geological condition is difficult, the tunnel must be provided in a sound ground conditions and it must be as short as possible.

2) The tunnel route must be determined to avoid as much as possible any fault area, fractured zones and soft foundation area. The existence of toxic or explosive gas must be investigated. Counter measures against fault areas, fractured zones and soft foundation area must be taken if the tunnel must be driven through faults, fractured and soft foundation areas.

3) Counter measures to prevent people from falling into the tunnel and to prevent siltation and sedimentation are necessary in the design of inlets to tunnels. Constructions of adits and shafts may be considered, in order to shorten the construction period and to minimize construction costs.

4) When there are farmland and housing areas where ground water is used for irrigation and domestic use, existing wells for irrigation and domestic use may dry up. In such cases sufficient investigation such as the position, volume, quality of groundwater must be made on conditions before the commencement of construction, and counter measures against such problems must be established.

(3) Culvert

A culvert is employed where a slope is so high in the case of an open channel, an open channel is unstable in the structural condition due to uplift pressure or uneconomical. A tunnel is difficult to employ due to small earth cover thickness when a canal crosses a railway, road and levee.

1) A culvert must be as short as possible as far as geological conditions permit.

2) Since a culvert is generally buried the ground and the stability of the culvert is not influenced by the rise of the groundwater table, it might be more economical than an open channel, which required a deep cut.

3) The depth necessary for burying a culvert must be determined, taking into consideration the water level required from hydraulic study, earth cover necessary for the purpose of the land use, etc.

4) When a culvert crosses a railway, river or road, the intersection angle must be right angle as far as possible. The jacking method might be lower in the total construction cost than the open-cut method. Therefore, a culvert

across a trunk road or railroad should be designed through a comparative study, taking into account the most appropriate construction method including temporary works.

5) If a culvert crosses urban and suburban areas, the construction cost of the conduit might be lower than that of an open channel since the culvert does not require facilities such as to prevent accidents and for disposal of water materials.

6) If a culvert is deeply buried, then the construction of the culvert may affect environmental conditions such as fluctuation of the groundwater table. Therefore, the adequacy of the route and counter measures against such environmental changes must be studied through environmental investigation.

(4) Siphon/aqueduct

A siphon and aqueduct may be employed to convey water across rivers, railroads, roads and valleys where the construction of an open channel maybe difficult or where the construction cost of the siphon/aqueduct is lower than that of an open channel due to shorter length of the route. The selection of either siphon/aqueduct or open channel must be made by taking into consideration the topography, geology, hydraulic conditions, stability and construction costs. The following points need to be considered for the selection of a siphon/aqueduct or an open channel:

1) Since the construction cost of a siphon/aqueduct is three to four times higher than that of an open channel, the siphon/aqueduct must be as short as possible.

2) A siphon barrel may be laid on the ground or buried. If part of the siphon barrel is replaced by an aqueduct or tunnel due to topographic or geological conditions, the construction cost of such a combined siphon may be lower than that of a siphon. If conveying water across a deep valley of hard geological foundation is required, a piped aqueduct above the flood water level might be more economical than a siphon. If the foundations at the inlet and outlet of the siphon is of rock formation and the valley is very deep, a combined use of tunnel and siphon may be more economical that a siphon.

3) When a siphon or aqueduct crosses such important facilities as road, river and railway, the intersectional angle should be as near as possible a right angle.

4) In general, the inlet and outlet of a siphon or aqueduct should not be constructed in a banking section for the sake of stability of the structure

and when such cases are required counter measures that are necessary not to avoid weak point in a canal system.
5) Determination of earth cover for a siphon is according to that of a culvert.

(5) Drop/chute

Where there is extra water head in the canal, drop/chute structures must be provided in the canal for the stability of the canal. The location and type of drop/chute structures must be determined through comparative design with regard to the stability and cost of the entire canal system. The following points need to be considered in the design:
1) If a canal gradient is steep, then the flow velocity will be high and the canal might easily be subject to scouring and erosion according to the conditions of its lining surface material. The design of the drop/chute must be carried out through a comparative study with regard to the location and type of the drop/chute taking into consideration lining materials and flow velocities for scouring and erosion.
2) Where the ground slopes are relatively steep, a canal slope must be determined through alternative studies taking into consideration canal structures and allowable velocities of various lining materials, and canal slope must be amended by the provision of the drop/chutes. When a velocity approaches the critical velocity, the surface of the flow becomes unstable. Excessive high velocity must not be occurred particularly in irrigation canals. If the construction of a regulating reservoir or farm pond is required, then extra water head might be effective for a canal system. Therefore, the use of this water head must be considered in the overall design.
3) Where a drainage/irrigation canal system is designed for a mountainous area, it is possible, due to relatively steep canal slopes, to secure stability of the entire canal system and to save construction costs by providing appropriate drops or chutes in the drainage/irrigation canal and to amend canal slopes. Where a lateral drainage canal joins the main drainage canal, the confluence might be provided at the dissipater of a drop/chute to reduce costs as possible.
4) When a drop/chute is provided nearby a housing area, inconveniences to inhabitants such as vibration, noise and splash from the drop/chute must be avoided as much as possible.

3.1.7 Particulars to be considered in Selection of Canal Route and Structures

For canal alignment, canal facilities or canal types consisting a canal system must be determined under the limited requirements such as curvature, longitudinal slope and earth cover of a canal taking into consideration topographical or other field condition.
The selection of canal route and structures must be determined so as to satisfy the following requirements:

① Minimum radius of canal curvature,
② Limits of longitudinal slope and curve of canal,
③ Minimum covering depth of soil

However, these requirements may be disregarded where they are inevitable due to topographical or other field conditions.

(1) Minimum radius of curve

For hydraulic reasons canal must be as straight as possible. Where a curve is required in the route, the following radius of curve must be provided:
1) Open channel and box culvert
(a) Radius of curve of the canal canter line should be more than ten times the water surface width of the canal except in a drainage canal.
(b) If metal forms are to be used in an open channel or box culvert, then the radius of curve should be more than 30 m.
2) Tunnel (reference)
(a) If a tunnel lining is made by using slide forms, the minimum radius of curve is calculated according to the following forms:

$$R = \frac{L_f^2}{8d_e} \quad \text{........................(F. 3.1)}$$

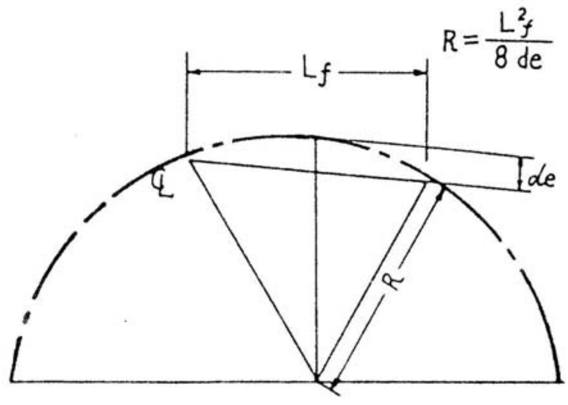


Fig. 3.3 Minimum radius of curve

Where:

R: Minimum radius of curve of tunnel center line (m)

L_f: Length of slide form (m)

d_e: Middle ordinate of circular length (m)

usually about	0.05 m
maximum	0.1 m

(b) When a tunnel is excavated using the shield method, the minimum radius of curve varies with ground conditions, excavated cross section, length of shield, construction method and shield structures. Generally, the minimum radius is determined by the length of shield and excavated cross section as follows:

$$R = m \cdot D \quad \text{or} \quad R = n \cdot L \quad \text{......................(F. 3.2)}$$

Where,

R : minimum radius of curve along the center line of a tunnel

m: min. 30 (usually about 50)

n : min. 20 (usually about 100/3)

D : diameter of excavated tunnel section (m)

L : length of shield (m)

(c) In case of a tunnel is excavated by using tunnel drilling machine, the minimum radius of the curvature is as follows taking into consideration past construction records:

$$R = m \cdot D \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (F.\ 3.3)$$

Where,
m : about 30 in Robinson Wallmeyer type
D : diameter of excavated tunnel section (m)

3) Siphon
(a) Since internal water pressure acts on the siphon barrel, a siphon must be designed so as to satisfy the allowable bend-angle in accordance with joint structures of the barrel. Generally, water tightness is maintained by using a bent pipe without bending the barrel.
(b) In order to maintain stability and water tightness of the siphon barrel, a thrust block is often provided on the bending point of the siphon with a large deflection angle to observe eccentric load.
(c) If a siphon is to be constructed by using in-site concrete, then radius of curve of a box culvert is applied to the siphon. If the water pressure is very high, then particular consideration must be given to the joint structure of the siphon.

(2) Limitation of longitudinal slope and curve
The slope of a water conveyance facility is generally determined, taking into consideration the topographic slope and required water level. If an allowable maximum slope is provided to the facility, then the cross-section area of the facility is small and its costs are low. However, when the velocity increases excessively, scour and turbulent flow may be caused. Appropriate hydraulic and structural limitations are, therefore, necessary for the slope and velocity. The allowable velocity must be determined so that scour and erosion may be prevented in accordance with lining materials. If velocity varies and is over the limitation with the fluctuation of discharge in such case of drainage canals, the canal must be designed as to be stable against scour and erosion by taking into account past similar designs. An irrigation canal of an open channel type must be designed so as to be hydraulically stable and to be adequate for diversion works. Excessively low velocity is not appropriate for an irrigation and drainage canal since sand sedimentation or growths of aquatic plants may occur. This limitation must be also considered in the determination of the canal route and in the allotment of the water head. If the velocity is lower than the allowable minimum velocity, then counter

measures must be established against it such as construction of special facilities to prevent sand from intruding into the canal.

(3) Minimum earth cover

Appropriate earth covering shall be secured for those facilities such as culverts, siphons, pipes, etc., buried underground according to purposes including protection of structures (reduction of the load, frost heaving, etc.), prevention of floatation due to the groundwater, utilization of the ground surface, etc. Generally, the following are standard values for the minimum earth covering thickness for water conveying facilities using pipes such as pipelines.

1) Crop lands:

0.6 m or greater + the depth of cultivated soil (Conditions of cultivation and laying conditions of culverts and siphons shall be considered.)

However, the minimum buried depth for the pipeline within farm fields which diameter is 300 mm or less can be 0.3 m or greater.

2) Under public roads:

1.2 m or greater. However, it shall be determined based on consultations with the road administrator.

3) It shall be 1.0 m or greater under farm roads and private roads in consideration of installation of appurtenant works, etc.

4) Under rivers: 2.0 m or greater.

5) Under forest: 0.6 m or greater.

6) In cold districts, the depth shall be set to prevent freezing,

7) These standard values shall be applied to box form culverts and siphons.

In case of reduction or elimination of the minimum earth covering thickness described above, sufficient consideration shall be provided to prevent the safety of structures from being compromised even under concentrated loads such as vehicle wheels. Also, it is necessary to determine with careful consideration for the water management aspects as well.

3.2 Hydraulic Design

3.2.1 General

The hydraulic design of canal must be made for the design discharge. Reviews must be also made of possible circumstances in which other design discharges are related to meet the requirements of other facilities in the canal system. Further, the hydraulic consistency in a series of structures of the canal system is maintained in the hydraulic design.

(1) Hydraulic design of canals

The hydraulic design of canals must be made for the design discharge and to secure hydraulically purposed designed level, and the design of respective facilities in the canal system must be made for not only the design discharge but also other discharges so that their purposes and functions can fully be fulfilled. Discharges other than the design discharge are the most-frequent discharge and the minimum discharge for an irrigation canal, and in a drainage canal, discharge to study low water revetment, and discharges that may make troubles to important canal facilities.

1) Irrigation canal

(a) Most-frequent discharge

This discharge is used to confirm the flow condition in examining the minimum allowable velocity in the canal facilities.

(b) Minimum discharge

This discharge is used to examine the functions of diversion works and related facilities for water level regulation when the discharge decreases.

2) Drainage canal

(a) Discharge to study low water revetment etc.

Probable discharge with a return period of one year or two is usually applied for the revetment planning of drainage canals. The water level related with this discharge is used for the structural design of the drainage canal such as the determination of the revetment height, etc. This discharge is also used to check the function of a drop. In addition, the minimum allowable velocity in the drainage canal is usually examined by use of the probable discharge with a return period of one year or two.

(b) Discharges that may make troubles to important canal facilities

When corresponding damages are expected from the scale of facilities, discharges, which flow the full section of a canal, should be studies to examine flow conditions and capacity of a canal and be used to study structural conditions, alignment of facilities and countermeasures for expected troubles.

(2) Establishment of the hydraulic consistency in the canal design

The establishment of the hydraulic consistency in the canal design is one of the basic concepts of this manual. However, such consistency does not mean simple and uniform standardization but comprehensive technical judgment from the viewpoints to secure function, safety and economy in the canal system.

3.2.2 Allowable Flow Velocity

The design velocity of canal must be determined within the limits of two factors: the minimum allowable velocity which produces neither deposit of sand and earth nor growth of water weeds, and the maximum allowable velocity which produces neither erosion if canal component materials by the flow not hydraulically unsafe conditions of flow in the canal.

(1) Determination of the design velocity

Determination of the design velocity is one of the important factors for the design of canals and the cross sections of canal structures. The design velocity is determined between the minimum and maximum allowable velocity taking into consideration the functions and structure of a canal.

(2) Minimum allowable velocity

It is not easy to determine a proper value of the minimum allowable velocity because there are under finable factors, which place restrictions on the minimum allowable velocity. It is obtained so as not to produce sand deposits and hinder the flow capacity of the canal by the presence of waterweed. In general, it is recognized that sand deposits are not produced at a mean velocity of 0.45-0.9m/sec in a canal where the particle size of suspended sediment is not larger than silt, and that waterweeds hindering the flow capacity of the canal will not grow when the mean velocity is more than 0.7m/sec. Therefore, it is approvable that velocity for the most–frequent discharge and the discharge to study low water revetment should not be lower than the above values when applying them to irrigation and drainage canals. The velocity in tunnels, culverts and siphons must be

larger than those in the open channels because when sand deposits are produced in these facilities, their flow capacities become constructed and it is difficult to remove such deposits.
The following velocity rations in tunnel, culvert and siphon are generally applicable:

Tunnel, Culvert – more than 1.3 times the velocity in open channel

Siphon – more than 1.5 times the velocity in open channel

(3) Maximum allowable velocity

Since the maximum allowable velocity varies remarkably with materials used for canals and is unclear, experiences and other examples must be considered. Limited velocities found to be applicable to different types of materials used for canals are given in Table 3.1. The maximum allowable velocity is required to be under 1.5 times of the value mentioned above for wasteway, spillway in an irrigation canal and for structures used temporary. In the case of a drainage canal, the above value is applied to discharges to study low water revetment.

Table 3.1 Maximum Allowable Velocity

Type of material	Velocity (m/s)	Classification	Velocity (m/s)
Sandy soil	0.45	Thick concrete (approximately 18 cm)	3.00
Sandy loam	0.60	Thin concrete (approximately 10 cm)	1.50
Loam	0.70	Asphalt	1.00
Clayey loam	0.90	Block cavity wall (buttress pier less than 30 cm)	1.50
Clay	1.00	Block cavity wall (buttress pier 30 cm or larger)	2.00
Sandy clay	1.20	Block mortar masonry	2.50
Soft rock	2.00	Reinforced concrete pipe	3.00
Semi-hard rock	2.50	Steel pipe, ductile cast iron pipe	5.00
Hard rock	3.00	Petrochemical products group (polyvinyl chloride pipe, reinforced plastic composite tube)	5.00
		Reinforced concrete secondary product canal (excluding fence culvert)	3.00

Notes: 1. The maximum allowable velocity is a value determined mainly by structural durability of the material of the canal structure against scour and wear. Specifically when a velocity close to the maximum allowable velocity value is used, it is necessary to study the hydraulic stability (especially regarding waves, water level rise at the cross section transition point, air entrapment in pipes, etc.).

2. Maximum allowable velocities for structures such as wasteways/spillways that are part of the canal and convey temporary flows shall be equal to or less than 1.5 times of values listed in the table above.

3. In cases of drainage canals, the value equal to or less than 1.5 times of values in this table shall be applied to discharges (1-year or 2-year probability discharge) to

study the low water revetment. However, such value shall not also exceed values in this table at the time of 185-day water discharge or firm drainage discharge during irrigation season. Additionally, this table is not applicable to cases where appropriate erosion protections such as bed protection, etc., are provided for the subject facility in areas such as chutes, steep slope drainage canals, etc., or where structural members are reinforced by means such as increasing concrete thickness or reinforcing bars, or where the drainage canal is as large as a river. In such cases, the maximum allowable flow velocities shall be determined by referring to the structure and topography/geology of the subject canal as well as similar case examples.

4. The maximum allowable flow velocities for cast-in-place concrete structures whose member thickness is 13 cm or larger shall be 3.0 m/s or less. Also, values of thick concrete or thin concrete in the above table may be applied to the maximum allowable flow velocities for plain concrete structures and for thickness between 10 cm and 18 cm, the value may be determined by proportional distribution.

(4) Considerations for determination of design velocity

Hydraulic conditions in a canal must be carefully examined in determining the design velocity of an irrigation canal. Under the nearly critical flow conditions, the water surface in the canal tends to become unstable, producing waves which do not vanish quickly. These cause a lowering in the efficiency of the canal function. It is generally recognized that the stability of flow in the canal is controlled by the velocity, though it can vary owing to many factors such as discharge, velocity, change of cross-section area, bends, etc. A velocity of less than <u>two-thirds of the critical velocity (Froude number: 0.54)</u> may be expected to stabilize the water surface in the canal. Accordingly, the above velocity must be applied to conditions under sub-critical flow in the irrigation canal. If a velocity larger than the above has to be adopted for any reason, necessary measures must be taken to heighten the canal wall and to provide special designs for division works, drops, etc. taking into consideration the water surface fluctuation, eccentric water surface at bend points of a canal. In addition, for facilities under super-critical flow conditions such as chutes, etc., the table is not applied and special attention must be given to the internal friction in the canal and hydraulic effects due to the changes in the cross-section of the canal such as enlargement, contraction and others.

3.2.3 Calculation of Mean Velocity

Dimensions of the cross section of a canal are determined, in principle, from the design discharge calculated by mean of velocity formula. The calculation of the uniform flow velocity must in principle be made according to the Manning's mean velocity formula for an open channel type canal and Hazen William's formula for a pipe line type canal.

(1) Discharge of canal

The discharge of the canal is calculated using the following formula:

$$Q = A \cdot V \quad \text{.................................} \quad \text{(F. 3.4)}$$

Where, Q: Discharge (m^3/s)

A: Cross-section area (m^2)

V: Mean velocity (m/s)

(2) Mean velocity formula of open channel type canal

The mean velocity of an open channel type canal in the above formula is calculated according to the Manning formula as a rule.

$$V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2} \quad \text{.................................} \quad \text{(F. 3.5)}$$

Where, V: Mean velocity (m/s)

n: Coefficient of roughness

R: Hydraulic radius (m) (R=Cross-section area (A)/Wetted perimeter (P))

P: Wetted perimeter (m)

I: Hydraulic gradient (canal bed slope)

In this manual, this formula is applied to mainly open channels or culverts and siphons partially included in an open channel type canal. For non-uniform flow condition, the energy gradient is substituted for the canal bed slope in this formula.

(3) Coefficient of roughness

1) Selection of coefficient of roughness

Selection of the coefficient of roughness is very important in use of the Manning formula, and therefore careful consideration is required in determining the coefficient on various influencing factors such as surface roughness, vegetation, bends, cross-section area and shape, velocity, hydraulic radius, sediment, scour, suspended materials, canal conditions of operation and maintenance stage, etc. In canals constructed with the same materials, the coefficient of roughness tends to become larger in the case of extremely slow velocity or small hydraulic radius. Standard values in the table are generally applied to design. It must also be considered that the smoothness of the internal section of the canal decreases gradually due to friction and scour, etc. are produced by the flow in the canal, and the growth of aquatic plants.

Table 3.2 Values for coefficient of roughness n

① Lining, retaining walls, tunnels, culverts, siphons, and aqueducts

Materials and conditions of canals	Coefficient of roughness		
	Minimum value	Standard value	Maximum value
Concrete (cast-in place flume, culvert, etc.)	0.012	0.015	0.016
Concrete (shotcrete)	0.016	0.019	0.023
Concrete (with precast flume, pipe, etc.)	0.012	0.014	0.016
Concrete (reinforced concrete pipe)	0.011	0.013	0.014
Concrete block masonry	0.014	0.016	0.017
Cement (mortar)	0.011	0.013	0.015
Asbestos cement pipe	0.011	0.013	0.014
Steel (locked bar or welded)	0.010	0.012	0.014
Steel (rivet)	0.013	0.016	0.017
Smooth steel surface (not painted)	0.011	0.012	0.014
Smooth steel surface and pipe (painted)	0.012	0.013	0.017
Corrugated surface (steel sheet)	0.021	0.025	0.030
Cast iron (not painted)	0.011	0.014	0.016
Cast iron sheet and pipe (painted)	0.010	0.013	0.014
Chloride vinyl pipe		0.012	
Reinforced plastic		0.012	
Ceramic pipe	0.011	0.014	0.017
Earth lining		0.025	
Asphalt (smooth surface)		0.014	
Asphalt (rough stone)		0.017	
Masonry (rough stone wet masonry)	0.017	0.025	0.030
Masonry (rough stone dry masonry)	0.023	0.032	0.035
Wood (wooden gutter)	0.010	0.012	0.014
Wood (lined in thin layer, treated with creosote)	0.015	0.017	0.020
Rock tunnel with no lining on overall cross-section area	0.030	0.035	0.040
Rock tunnel with no lining except concrete placed on the bottom	0.020	0.025	0.030
Vegetation coverage (sodding)	0.030	0.040	0.050

② Canals constructed by excavation or dredging

Materials and conditions of canals	Coefficient of roughness		
	Minimum value	Standard value	Maximum value
Earthen canals, uniform and straight			
1) No weeds (immediately after completion of the canal)	0.016	0.018	0.020
2) No weeds (after the canal has been exposed to weather)	0.018	0.022	0.025
3) Gravel (no weeds)	0.022	0.025	0.030
4) Few weeds with short grasses	0.022	0.027	0.033
Earthen canals, curved and non-uniform			
1) No vegetation coverage	0.023	0.025	0.030
2) Some weeds	0.025	0.030	0.033
3) Dense growth of weeds or water weeds	0.030	0.035	0.040
4) The bottom is earth and the side walls are covered by rubble stones	0.028	0.030	0.035
5) The bottom is covered by stones, and the side walls are covered by weeds	0.025	0.035	0.040
6) The bottom is covered by cobble stones, and the side walls have no weed	0.030	0.040	0.050
Drag line excavation and dredging			
1) No vegetation coverage	0.025	0.028	0.033
2) Some shrubs on shore	0.030	0.050	0.060
Rock excavation			
1) Smooth and uniform	0.025	0.035	0.040
2) Irregular	0.035	0.040	0.050

③ Natural flow canals

Materials and conditions of canals	Coefficient of roughness		
	Minimum value	Standard value	Maximum value
Small canals on flat land			
1) No weed and straight. No fracture or deep water spot when the high-water level is reached	0.025	0.030	0.033
2) Same as above, but a lot of stones and weeds	0.030	0.035	0.040
3) No weed, but meandering. Some shoals and deep water spots	0.033	0.040	0.045
4) Same as above, but some stones and weeds	0.035	0.045	0.050
5) Same as above, but low-water level and few changes in slopes and cross sections	0.040	0.048	0.055
6) Same as the line item 4) above, but more stones	0.045	0.050	0.060
7) Weeds and deep spots in mild flow sections	0.050	0.070	0.080
8) Section with thick vegetation of weed. Many deep water spots and trees	0.075	0.100	0.115
Canal in mountainous land, no plant in the canal. River banks are steep. Trees and shrubs along river banks are immersed in the water when the high-water level is reached			
1) River bed is covered by cobble stones and gravels	0.030	0.040	0.050
2) River bed is covered by large cobble stones	0.040	0.050	0.070
Large canals			
1) Regular cross section without large cobble stones or shrubs	0.025		0.060
2) Irregular and rough cross section	0.035		0.100

2) Effects of bends, sedimentation and vegetation on coefficient of roughness

The following factors have an effect on the coefficient of roughness.

(a) Canal bends: The coefficient of roughness (n) increases when a canal meanders because such meandering produce losses and sand deposits in the canal. In the case of low velocity, increases of n may be neglected. In general, n increases about 0,002 as an allowance for canal bend losses when the canal has curves, and n in a natural meandering channel increases by 30%.

(b) Sedimentation: n is affected by sand deposits in a canal and increases significantly in the case of non-uniform sediments such as sandbars and sand ripples.

3) Coefficient of compound roughness

If the Manning formula is applied to the cross-section of the canal having different coefficients of roughness for some parts of wetted perimeter (P), the velocity must be calculated using a compound roughness coefficient for the whole wetted perimeter. (Fig 3.4, Table 3.3)

The compound roughness coefficient is estimated by the following formula:

$$n_i = \left\{ \frac{1}{\sum P_i} \left(P_1 \bullet n_1^{3/2} + P_2 \bullet n_2^{3/2} + \dots\dots\dots + P_5 \bullet n_5^{3/2} \right) \right\}^{2/3} \quad \text{................................ (F. 3.6)}$$

It is not advisable to calculate the discharge using the coefficient of compound roughness if the water depth at high water is small in drainage canals, rivers, etc. In this case, it is usual to make calculation for the subdivision of the cross-section area as shown in Fig. 3.5. The interface of the subdivision is not regarded as wetted perimeter.

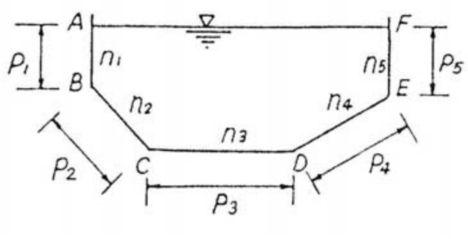


Fig. 3.4 Wetted Perimeter

Table 3.3 Compound roughness coefficient

Zone	Coefficient of roughness	Length of wet perimeter
AB	n_1	P_1
BC	n_2	P_2
CD	n_3	P_3
DE	n_4	P_4
EF	n_5	P_5
Total	n_i	ΣP_i

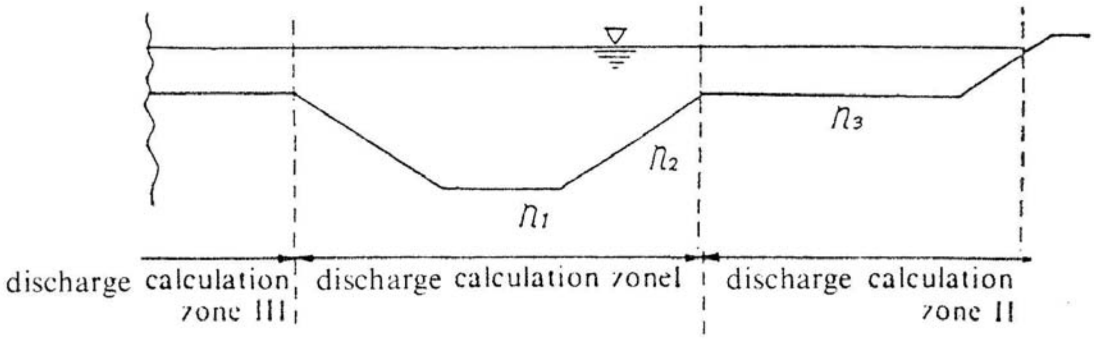


Fig. 3.5 Subdivisions of Cross-Sectional area

(4) Determination of cross-section area of a uniform flow canal

The water depth of a uniform flow canal can be obtained by trial calculations using the Manning formula. Alternatively, other various calculation methods may be suggested for convenience' sake such as $A \cdot R^{2/3} = n \cdot Q\sqrt{I}$, obtained by Manning's mean velocity formula (F. 3.5) and a formula (F. 3.4). The graphic solution method is also useful for obtaining answers easily for a complicated cross-section area. The most appropriate method must be selected after taking into consideration factors, which affects the determination of the cross-section area of the canal. For a canal with a relatively gentle slope from the topographic point of view, the maximum possible canal slope may be an important factor for determination of the cross-section area of the canal. Whereas when topographic slope becomes steep and velocity high speed, it is necessary to modify the canal slope by providing drops for the prevention of high velocity in the canal.

(5) Mean Velocity Formula of a Pipeline Type Canal

The mean velocity must, in principle, be calculated by Hazen William's formula for a pipeline type canals.

$V = 0.355\, C \cdot D^{0.63} \cdot I^{0.54}$ .... (F. 3.7)

Where V : Mean velocity (m/s)

C : Coefficient of velocity

D : Diameter of pipe (m)

I : Hydraulic gradient

I in the formula (F. 3.7), is hydraulic gradient, and can be obtained by the next formula.

$$I = h_f / L \quad \text{.......................................(F. 3.8)}$$

Where h_f : Loss head for friction (m)

L : Length of pipeline (m)

Values in Table 3.4 are applied to design as coefficient of velocity.

Table 3.4 Coefficient of velocity

Pipe (Inside condition)	Coefficient of velocity		
	Maximum value	Minimum value	Standard value
Cast iron pipe (not painted)	150	80	100
Steel pipe (not painted)	150	90	100
Coal-tar painted pipe (Cast iron)	145	80	100
Tar epoxy painted pipe (Steel)			
Ø 800 ~	—	—	130
Ø 700 ~ 600	—	—	120
Ø 500 ~ 350	—	—	110
Ø 300 ~	—	—	100
Mortar lining pipe (Steel, Cast iron)	150	120	130
Reinforced concrete pipe	140	120	130
Pre-stressed concrete pipe	140	120	130
Chloride vinyl pipe	160	140	150
Polyethylene pipe	170	130	150
Reinforced plastic pipe	160	—	150

Note: C = 140, is taken for the pipes having diameters less than 150mm in principle.

3.2.4 Non–uniform Flow

Flow conditions in the canal reaches where the canal section of flow is not uniform due to changes in the canal section or backwater above and down a weir are analyzed by using the non-uniform flow calculation method.

(1) Basic equation of non-uniform flow (reference)

Non-uniform flow occurs when the water level and velocity are constant in time scale but changes at some position in an open channel. Such flow is generally found at place where canal section change such as connecting canals, division works, drops, etc. or at places affected by backwater due to changes of canal slopes and provision of weirs.

The formula for non-uniform flow is expressed as follows:

$$-i + \frac{dh}{dx} + \alpha \frac{Q^2}{2g} \cdot \frac{d}{dx}\left(\frac{1}{A^2}\right) + \frac{n^2 V^2}{R^{4/3}} = 0 \quad \text{................................ (F. 3.9)}$$

Where i : Canal bed slope
h: Water depth (m)
X: Length toward the downstream along the canal bed (m)
Q: Discharge (m^3/ s)
A: Cross-section area (m^2)
g: Acceleration due to gravity (m/ s^2)
α: Coefficient of energy correction (α = 1.1, is used in general but α = 1.0 may be used for simple calculation)
n: Coefficient of roughness
V: Mean velocity (m/s)
R: Hydraulic radius (m)

The water surface of a non-uniform flow is traced by the solution of the above mentioned basic equation of non-uniform flow, or step calculation method and graphic solution method. The calculation starts from the point, where hydraulic conditions are decided, to upstream in the case of subcritical flow condition and to downstream in the case of supercritical flow condition respectively. It must be examined that the control sections, where conditions in a canal change flow, may occur, and when the central section is expected, calculation should be start from this section. The water depth of the control section, this is the critical depth, is obtained by calculation mentioned below:

(2) Critical depth

"Critical depth " is produced at places where there are sudden changes in canal slope such as at drops, chutes, etc., and is the boundary between subcritical and supercritical flow. The critical depth, water depth of boundary of subcritical and supercritical flow, may be suggested by various methods, but is defined by the following equations: (F. 3.10) – (F. 3.12).

$$(Q^2/ (g A^3)) \cdot (dA/ dh) = 1 \quad \text{...............................(F. 3.10)}$$

or $$Q^2/ g = A^3/ T \text{ or } Q/ \sqrt{g} = A\sqrt{D} \quad \text{.............................(F. 3.11)}$$

or $$V^2 / 2g = D/2 \quad \text{.............................(F. 3.12)}$$

Where T: Width of water surface (m)
D: Hydraulic depth D = A/T (m)
Q: Discharge (m^3/ s)
g: Acceleration of gravity (m/s^2)

h: Water depth (m)
A: Sectional area of flow (m^2)

A cross section producing the critical depth is called the control section and influences upstream with subcritical flow and downstream with supercritical flow. The critical depth is obtained by three methods. In this guideline shows Algebraic method:

[Algebraic method]
If a canal has a simple geometric cross section, the critical depth (hc) can be obtained by the algebraic method using the above formula (F. 3.10) or (F. 3.11).
In a rectangular section, T = b, A = b·h
Therefore,

$$\frac{Q^2}{g} = \frac{b^3 \cdot h_c^3}{b} = b^2 \cdot h_c^3$$

$$hc = \sqrt[3]{Q^2 / gb^2} = \sqrt[3]{q^2 / g} = 0.467 q^{2/3} \quad \text{.................................. (F. 3.13)}$$

Where hc: Critical depth (m)
q : Discharge per unit width (m^3/s/m)

$$q = \frac{Q}{b}$$

b : Width of canal (m)
T : Width of water surface (m)
Q : Discharge (m^3/s)
g : Acceleration of gravity (m/s^2)

The following relation can also be found for the critical depth in a rectangular section.

$$hc = \frac{3}{2} \cdot H_c \quad \text{.................................. (F. 3.14)}$$

$$V_c = \sqrt{ghc} \quad \text{.................................. (F. 3.15)}$$

Where H_c: Specific energy in the control section (m)

V_c : Critical velocity (m/s)
g : Acceleration of gravity (m/s^2)

(3) Head losses and change of water level

1) Head losses (which should be) considered in hydraulic design

The following head loss must be considered in principle in the canal design.

① Head loss due to friction ,
② Head loss due to inflow or outflow,
③ Head loss due to change of canal section,
④ Head loss due to trash rack, and
⑤ Head loss due to pier.

The head loss due to canal curve is generally neglected. The change of the water level is expressed using the Bernoullis' formula as follows:

$$\Delta h = Z_2 - Z_1 = \sum h_i + \left(\frac{V_2^{\ 2}}{2g} - \frac{V_1^{\ 2}}{2g} \right) \text{.................................. (F. 3.16)}$$

Where Σh_i : Total head loss (m)
V_1 : Velocity in the upstream section (m/s)
V_2 : Velocity in the downstream section (m/s)
Z_1 : Water level in the upstream section (m)
Z_2 : Water level in the downstream section (m)
Δh : Difference in water level between the upstream and downstream sections (m)

If the canal section are nearly uniform or change gently and continuously in a certain distance, then the head loss may be considered to be from friction. In this case, the fall in the water surface due to friction will be equal to the canal bed slope. If the head loss due to inflow, outflow, sudden enlargement or contraction of the canal section is partially produced, the head loss due to the above factors represents the difference of water at the point where it occurs, although it is actually produced over a short distance and not at the point of occurrences. In addition, if the canal section changes continuously over some distance such as in transitional sections, the total head loss in this section is obtained by adding the head loss due to friction to the one due to change of section. In general, the water surface becomes low due to head losses, but in rare cases of enlargement of the downstream section, the water surface becomes higher. In irrigation and drainage canals, various head losses are produced by change of flow or turbulent flow due to many factors as mentioned above.

(4) Various head losses

1) Head loss due to friction

The calculation of head loss due to friction is made by using the Manning formula as shown below:

$$h_f = \frac{Q^2 \cdot l}{2} \cdot \left(\frac{n_1^{\ 2}}{R_1^{\ 4/3} \cdot A_1^{\ 2}} + \frac{n_2^{\ 2}}{R_2^{\ 4/3} \cdot A_2^{\ 2}} \right) = \frac{1}{2} \cdot \left(\frac{n_1^{\ 2} \cdot V_1^{\ 2}}{R_1^{\ 4/3}} + \frac{n_2^{\ 2} \cdot V_2^{\ 2}}{R_2^{\ 4/3}} \right) \cdot l \quad \text{.......................... (F. 3.17)}$$

Where
- Q: Discharge (m^3/s)
- h_f: Head loss due to friction (m)
- R: Hydraulic radius (m)
- V: Mean Velocity (m/s)
- A: Cross section area of flow (m^2)
- l: Distance calculated (m)
- n: Coefficient roughness

Number attached to R, A, n, V denote the Section I and II respectively.

2) Head loss due to inflow or outflow and change of water level

(a) Inflow

The head loss and change of water level due to inflow is calculated by the following formula in case of hydrostatic surface in which the velocity of inflow can be neglected.

$$h_{en} = f_e \cdot \frac{V^2}{2g} \quad \text{................................ (F. 3.18)}$$

$$\Delta h_{en} = h_{en} + \frac{V^2}{2g} \quad \text{................................ (F. 3.19)}$$

Where
- h_{en} : Head loss due to inflow (m)
- Δh_{en} : Change of water level (m)
- V : Mean velocity after inflow (m/s)
- g : Acceleration of gravity (m/s^2)

f_e : Coefficient of head loss due to inflow which is determined by slope of inlet as shown in Fig. 3.6.

If the velocity of inflow cannot be neglected, the head loss and change of water level due to such inflow is calculated by the formula, which is used in change of the canal section shown later 3.2.4 (5).

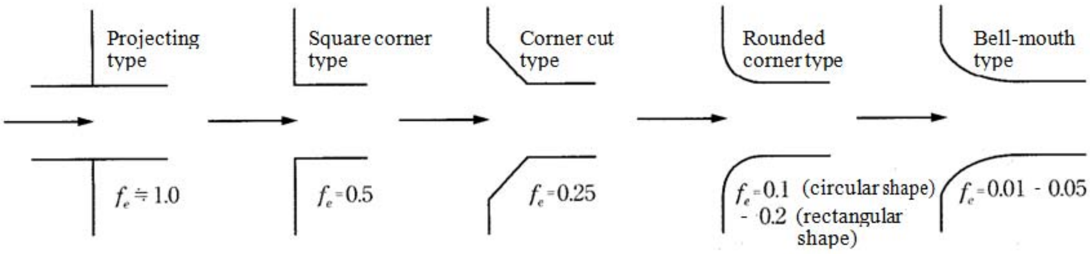


Fig. 3.6 Type of inlet transition and inlet head loss coefficient

(b) Outflow

Head loss and change of water levels due to outflow are calculated as follows:

$$h_{ou} = f_o \cdot \frac{V^2}{2g} \quad \text{.................................} \text{ (F. 3.20)}$$

$$\Delta h_{ou} = h_{ou} + \frac{V^2}{2g} \quad \text{.................................} \text{ (F. 3.21)}$$

Where h_{ou} : Head loss due to outflow (m)

Δh_{ou} : Change of water level (m)

V : Mean velocity before outflow (m/s)

g : Acceleration of gravity (m/s^2)

f_0 : Coefficient of head loss due to outflow which is generally taken to be 1.0 considering that all velocity energies in the canal are lost.

(5) Head loss and change of water level due to change of canal section

Head loss and change of water level due to change of the canal section are calculated as follows:

1) Gradual contraction

$$h_{gc} = h_c + h_f = f_{gc}\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) + I_m \cdot L \quad \text{.................................} \text{ (F. 3.22)}$$

$$\Delta h_{gc} = h_{gc} + (\frac{V_2^{\ 2}}{2g} - \frac{V_1^{\ 2}}{2g}) \quad \text{.................................} \quad \text{(F. 3.23)}$$

Where h_{gc} : Head loss due to gradual contraction (m)

h_c : Head loss due to gradual contraction of transition (m)

h_f : Head loss due to friction in transition (m)

Δh_{gc} : Change of water level (m)

V_1 : Mean velocity before gradual contraction (m/s)

V_2 : Mean velocity after gradual contraction (m/s)

g : Acceleration of gravity (m/s^2)

I_m : Mean hydraulic gradient in length of transition L

$$I_m = \frac{I_1 + I_2}{2}$$

I_1 : Hydraulic gradient before transition

I_2 : Hydraulic gradient after transition

L : Length of transition (m)

f_{gc} : Coefficient of head loss due to gradual contraction

f_{gc} varies with the shape of transition (See Fig. 3.8), and Table 3.5 indicating f_{gc} of the inlet and outlet transition to siphon, culvert and tunnel.

$$L = \frac{B - b}{2} \cdot \cot\theta \quad \text{.................................} \quad \text{(F. 3.24)}$$

Where L : Length of an open transition (m)

B : Water surface of an open channel (m)

b : Water surface of a closed transition, culvert or flume (m)

θ : Angle of construction (°) (generally less than 10°)

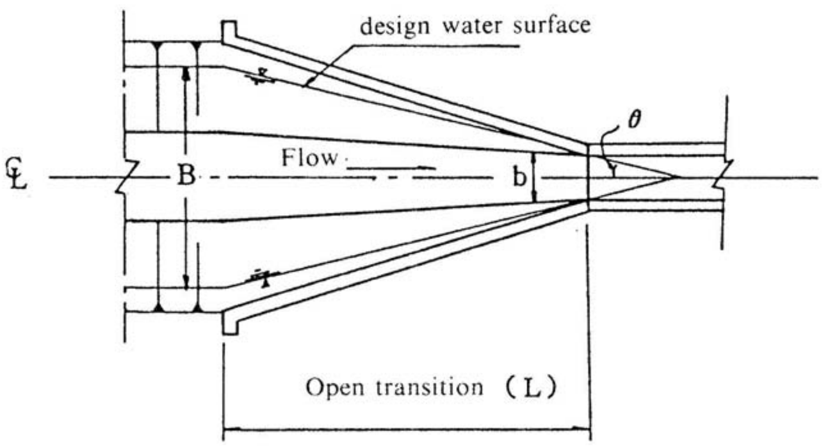
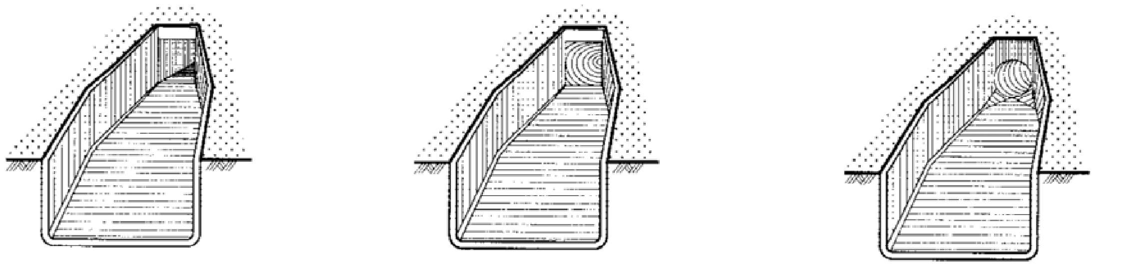


Fig. 3.7 Explanation for open transition

The values shown in Table 3.5 are some experimental values where the cross-section access of the flow of siphon, tunnel and conduit were relatively small compared with those of connecting open channels upstream and downstream. In the above experiment, the length of the transition was taken so that the angle of the side wall and center line of the canal was 12°30'. The head loss due to gradual contraction increases sharply as the angle of transition increases more than 12°30'.

Table 3.5 Head Loss Coefficient at Transition (Siphon, Culvert, Tunnel)

Condition of open transition formation change		Coefficient due to gradual contraction *fgc*	Coefficient due to gradual expansion *fge*	Number in Fig. 3.8
Straight line formation	A rectangular cross section is narrowed in transition to be connected with the rectangular opening at its opposite end.	0.10	0.20	(1)
Straight line formation	A rectangular cross section is narrowed in transition to be connected with the closed transition installed at its opposite end, or a fillet is installed on the open transition to make smooth connection with a circular opening.	0.20	0.30	(2)
Trapezoidal straight line formation	A trapezoidal cross section is transformed into a rectangular cross section by twisted wall surfaces in transition to be connected with a rectangular opening at its opposite end.	0.20	0.30	(3)
Trapezoidal straight line formation	A trapezoidal cross section is transformed into a rectangular cross section by twisted wall surfaces in transition to be connected with a closed transition installed at its opposite end, or a fillet is installed on the open transition to make smooth connection with a circular opening.	0.30	0.40	(4)
Trapezoidal bent formation	A trapezoidal cross section is transformed into a rectangular cross section by bent wall surfaces to be connected with a rectangular opening at its opposite end.	0.30	0.50	(5)
Trapezoidal bent formation	A trapezoidal cross section is transformed into a rectangular cross section by bent wall surfaces to be connected with a circular opening at its opposite end.	0.40	0.70	(6)

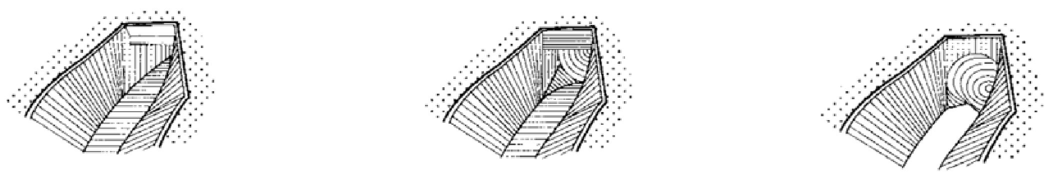


(1) Straight line formation

A closed transition installed is installed between the open transition and a circular opening to make smooth connection.

A fillet is installed to the open transition in front of the circular opening to make smooth connection with the circular opening.

(2) Straight line formation



(3) Trapezoidal straight line formation

A closed transition installed is installed between the open transition and a circular opening to make smooth connection.

A fillet is installed to the open transition in front of the circular opening to make smooth connection with the circular opening.

(4) Trapezoidal straight line formation

(5) Trapezoidal bent formation

(6) Trapezoidal bent formation

Fig. 3.8 Shapes of open transitions

2) Sudden contraction

$$h_{sc} = f_{sc} \cdot \frac{V_2^{\,2}}{2g} \quad \text{.................................. (F. 3.25)}$$

$$\Delta h_{sc} = h_{sc} + \left(\frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g}\right) \quad \text{.................................. (F. 3.26)}$$

Where h_{sc} : Head loss due to sudden contraction (m)
Δh_{sc} : Change of water level (m)
V_1 : Mean velocity before sudden contraction (m/s)
V_2 : Mean velocity after sudden contraction (m/s)
f_{sc} : Coefficient of head loss due sudden contraction (Refer to Table 3.6)
g : Acceleration of gravity (m/s^2)

Table 3.6 Head loss coefficient for sudden contraction

$\frac{A_2}{A_1}$	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	(1.0)
F_{sc}	0.50	0.48	0.45	0.41	0.36	0.29	0.21	0.13	0.07	0.01	(0)

Note: A_1: Cross-sectional area before sudden contraction (m^2), A_2: Cross-sectional area after sudden contraction (m^2)

3) Gradual enlargement

$$h_{ge} = h_e + h_f = f_{ge}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) + I_m \cdot L \quad \text{................................ (F. 3.27)}$$

$$\Delta h_{ge} = h_{ge} + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \quad \text{................................ (F. 3.28)}$$

Where h_{ge} : Head loss due to gradual enlargement (m)
h_e : Head loss due to gradual enlargement in transition (m)
h_f : Head loss due to friction in length of transition L (m)
Δh_{ge} : Change of water level due to gradual enlargement (m)
V_1 : Mean velocity before gradual enlargement (m/s)
V_2 : Mean velocity after gradual enlargement (m/s)
g : Acceleration of gravity (m/s^2)
I_m : Mean hydraulic gradient in transition

$$I_m = \frac{I_1 + I_2}{2}$$

I_1, I_2 : Hydraulic gradient before and after transition

f_{ge} : Coefficient of head loss due to gradual enlargement
(Refer to Fig. 3.8). Table 3.5 indicates f_{ge} of the inlet and outlet transition to siphon, culvert and tunnel.

4) Sudden enlargement

$$h_{se} = f_{se} \cdot \frac{V_1^{\,2}}{2g} \quad \text{.................................. (F. 3.29)}$$

$$\Delta h_{se} = h_{se} + \left(\frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g} \right) \quad \text{.................................. (F. 3.30)}$$

Where h_{se} : Head loss due to sudden enlargement (m)
Δh_{se} : Change of water level (m)
V_1 : Mean velocity before sudden enlargement (m/s)
V_2 : Mean velocity after sudden enlargement (m/s)
f_{se} : Coefficient of head loss due to sudden enlargement
$f_{se} = (1 – A_1/A_2)^2$ (refer to Table 3.7)
A_1 : Cross-section area before sudden enlargement (m^2)
A_2 : Cross-section area after sudden enlargement (m^2)
g : Acceleration of gravity (m/s^2)

Table 3.7 Head loss coefficient for sudden enlargement

$\frac{A_1}{A_2}$	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	(1.0)
f_{se}	1.0	0.81	0.64	0.49	0.36	0.25	0.16	0.09	0.04	0.01	(0)

3.3 Structural Design

3.3.1 General

In the structural design of a canal, the type of the structures, design condition, and the details of the structures are determined in consideration of the site conditions and the economic in compliance with the load applied to the structures, mechanical properties of the soil and meteorological conditions.

3.3.2 Load

The loads to be taken into consideration when designing a civil engineering structure are dead loads, water pressures, buoyancy or uplift pressures, earth pressures, vehicle loads, impact loads, crowd loads, track loads, seismic loads, wind loads, construction loads, temperature change, drying shrinkage and creep of concrete, frost heave pressures, and other loads. All of which must be examined in accordance with the importance of structure, type of structure, materials to be used, construction place, construction method, natural conditions, and other factors.

(1) Dead weight

In the calculation of dead weight, the standard unit weight of materials shown in Table 3.9 can be used; however, the actual unit weight obtained from experimentation or specified unit weight should be applied when available.

Table 3.9 Standard unit weight table

Material	Unit weight (kN/m^3)	Material	Unit weight (kN/m^3)
Steel, cast steel	77.0	Pre-stressed concrete	24.5
Cast iron	71.0	Granite	27.0
Aluminum alloy plate	27.5	Sandstone	26.0
Reinforced concrete	24.5	Soil (dry)	16.0
Plain concrete	23.0	Soil (wet)	18.0
Mortar	21.0	Soil (saturated)	20.0
Asphalt (for waterproofing application)	11.0	Soil (submerged)	10.0
Asphalt concrete pavement	22.5	Water	9.8
Wet masonry concrete block	22.5		
Dry masonry concrete block	19.5		

(Note)

Unit weight of material shall differentiate depending on each case; therefore, it is desirable that the actual unit weight should be clear in advance. However, much effort is required to carry out the investigation and experimentation to find the actual unit weight. It requires great technical experience and judgment to decide the optimum design figures from the result of such investigation and experimentation. In design, the figures shown in Table 3.9 can be applied in normal cases. When materials are not uniform in quality because of mixing and proportion, the Table shows the standard range. It is recommended to decide the design figures by taking into consider each of the design conditions.

Regarding the unit weight of concrete, when the actual weight of concrete used in large-scale facilities or sites such as dams are known the actual weight shall be used.

(2) Water pressure

The hydrostatic pressure acting perpendicular to the working face shall be obtained by using the equation (F. 3.31) (Fig. 3.9).

$$P = \gamma_w \cdot h \quad \text{.........................} \quad \text{(F. 3.31)}$$

Where p: Hydrostatic pressure (KN/m^2)

γ_w: Unit weight of water (KN/m^3)

h: Height from water surface to acting point (m)

h: Vertical distance from water surface to point of action (m)

h1: Vertical distance from water surface to canal bed (m)

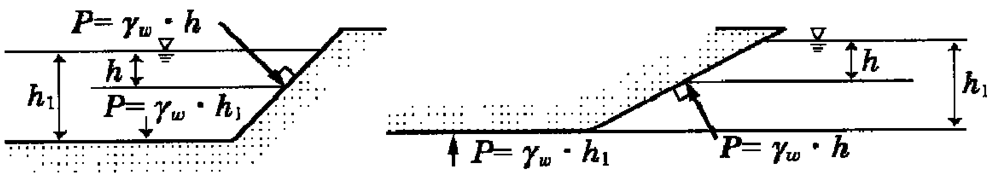


Fig. 3.9 Hydrostatic pressure at working face

(3) Buoyancy and up-lift

Buoyancy or uplift pressure which is assumed to act upward in vertical direction shall be taken into account in studying floating, overturning, sliding, etc., during a stability analysis of the structure, and shall not be taken into account in studying load bearing capacity of the ground. When the buoyancy or uplift pressure is considered as a load, the friction angle between the wall surface of structure and the soil may be taken into account.

(4) Earth pressure

Earth pressure is broadly divided into horizontal and vertical earth pressures. Horizontal earth pressure acting on the canal wall shall be obtained in accordance with the Rankin's and Coulomb's earth pressure formulas, or the Trial Wedge Method (refer to Table 3.20). The vertical earth pressure acting on a buried structure shall be obtained in accordance with the Vertical Earth pressure formula, or Marston's formula. In addition, the horizontal earth pressure acting on a buried structure shall be obtained in accordance with the Earth pressure at Rest formula, or the Rankin's and Coulomb's earth pressure formulas and Spangler's formula, etc.

(5) Vehicle load and impact load

1) Vehicle load

The magnitude, position, and dimension of vehicle load shall be in accordance with Fig. 3.10 and Table 3.10.

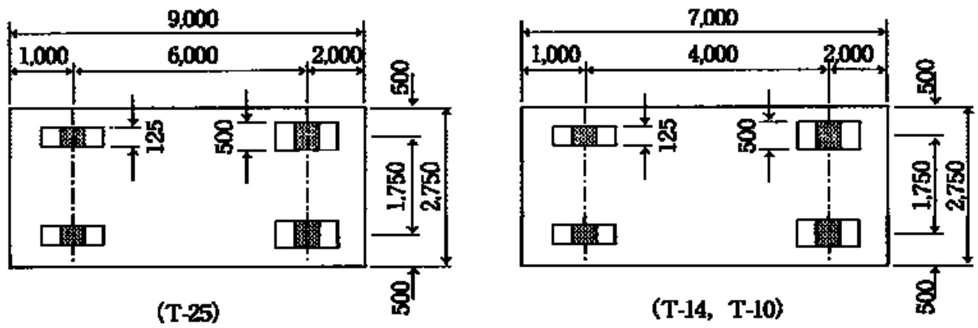


Fig. 3.10 Vehicle load (location and dimensions of wheel)

Table 3.10 Magnitude and dimension of vehicle load

Load	Total weight W (kN)	Front wheel load 0.1 W (kN)	Rear wheel load 0.4 W (kN)	Front wheel width b_1 (cm)	Rear wheel width b_2 (cm)	Wheel tread width a (cm)	Remarks
T-25	245	22.5	100	12.5	50	20	Standard Specifications for Highway Bridges
T-14	137	13.5	55	12.5	50	20	
T-10	98	10	39	12.5	50	20	

2) Applicable categories of vehicle load

The guideline for applicable categories based on the carriageway width is generally provided in accordance with Table 3.11.

Table 3.11 Applicable categories of vehicle load

Carriageway width (2 lanes)	Vehicle load (T)	Remarks
Less than 5.5	14 or less	Design vehicle load with carriageway width (2 lanes) of 5.5 m or less may be set to 25 tons depending on the situation.
5.5 or larger	25	

3) Impact load

Dynamic loads including vehicle load and track load would produce impact loads. However, the impact loads shall not be considered for crowd load. Table 3.12 shall be used as the standard for impact coefficients when impact load is considered.

Table 3.12 Standard values of impact coefficient ‘ i ’ (truck load) and Dynamic loads

Loading condition	Earth covering (m)	Impact coefficient i	Earth covering (m)	Dynamic load (kN/m^2)	Impact coefficient i
T-25	Less than 4.0	0.3	4.0 or greater	10	0
T-14	Less than 3.5	0.3	3.5 or greater	7	0
T-10	Less than 3.5	0.3	3.5 or greater	5	0

In the case of T-25 with earth covering of 4 m or greater, and in the case of T-14 or smaller with earth covering of 3.5 m or greater, the dynamic load values provided in Table 3.12 shall be uniformly distributed over the top slab surface.

4) Vertical load distribution and load strength

In principle, the vertical load shall be the uniformly distributed load spreading at an angle of 45°downward over the box culvert and pipe body only in vehicle’s traveling direction. In addition, the vertical load shall be uniformly applied over effective width of the vehicle in its lateral direction.

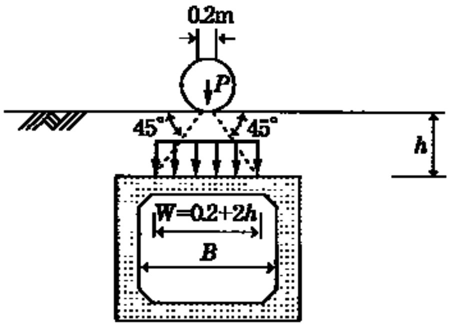


Figure 3.11
Load distribution in span direction

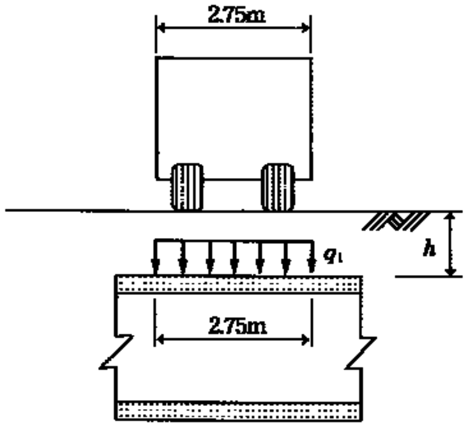


Figure 3.12
Load distribution in longitudinal direction (road transverse direction)

Selection of the vehicle load acting on box culvert shall be performed with due consideration so that the stress resultant generated in the box culvert section shall be calculated appropriately based on the directions of vehicle's travel and box culvert and on the physical relationship between the box culvert and the vehicle.

(a) Vehicle load acting on box culvert

The vehicle load acting on the top slab of box culvert shall be calculated under assumption that the front and rear wheel loads with tread width of 0.2 m are uniformly distributed spreading only in span direction at an angle of 45° downward. Also, a vehicle shall be loaded in the longitudinal direction (road transverse direction) without limitation.

a) Vehicle load in box culvert longitudinal direction (per unit meter)

$$P_w = \frac{2 \times \text{Wheel load}}{\text{Width occupied by vehicle}} \times (1+i) \quad \text{.............................. (F. 3.32)}$$

Where P_w: Wheel load per width occupied by vehicle (2.75 m) (kN/m)
i: Impact coefficient according to Table 3.12
Wheel load shall be according to Table 3.10

b) Load strength acting on box culvert top slab

$$q_1 = \frac{P_w \cdot \beta}{W} = \frac{P_w \cdot \beta}{2h + 0.2} \quad \text{.............................. (F. 3.33)}$$

Where q_1: Load strength due to wheel load (kN/m^2)
P_w: Wheel load per width occupied by vehicle (kN/m)
β: Reduction coefficient of stress resultant (Table 3.13)
W: Distribution width of rear wheel load (m)
h: Earth covering (m)

Table 3.13 Reduction coefficient of stress resultant

Loading condition	Reduction coefficient ; β	
T-25	1.0 Earth covering h is less than 1 m, and inside cavity width is 4 m or larger.	0.9 (Other than noted in the left)
T-14 or smaller	1.0	

c) Consideration of front wheels effect

When the rear wheel load P1 is loaded on the span center, the distributed load due to the front wheel load P2 shall be loaded (Fig. 3.13). However, when the earth covering is 4 m or larger (or 3.5 m or larger in the case of T-14 or smaller), it shall not be considered in the same manner as the rear wheels.

$$W_2 = \frac{B_1}{2} + h - (L - 0.1)\ (\mathrm{m}) \quad \text{............................ (F. 3.34)}$$

The vertical load q_2 (kN/m2) due to the front wheels shall be obtained in a manner similar to q_1.

$$q_2 = \frac{2 \times P_2}{\text{Width occupied by vehicle}} \times (1+i) \times \frac{1}{W_2 + W_3} \quad \text{.............................. (F. 3.35)}$$

Where P_2: Front wheel load (kN)
$W_2 + W_3 = 2\,h + 0.2$

The distribution widths W2 and W3 shall be blocks as shown in Fig. 3.13, and the effect due to W3 shall be considered as horizontal load (Ph = q2 • Ko).

Ko: Coefficient of earth pressure at rest (Ko = 0.5)

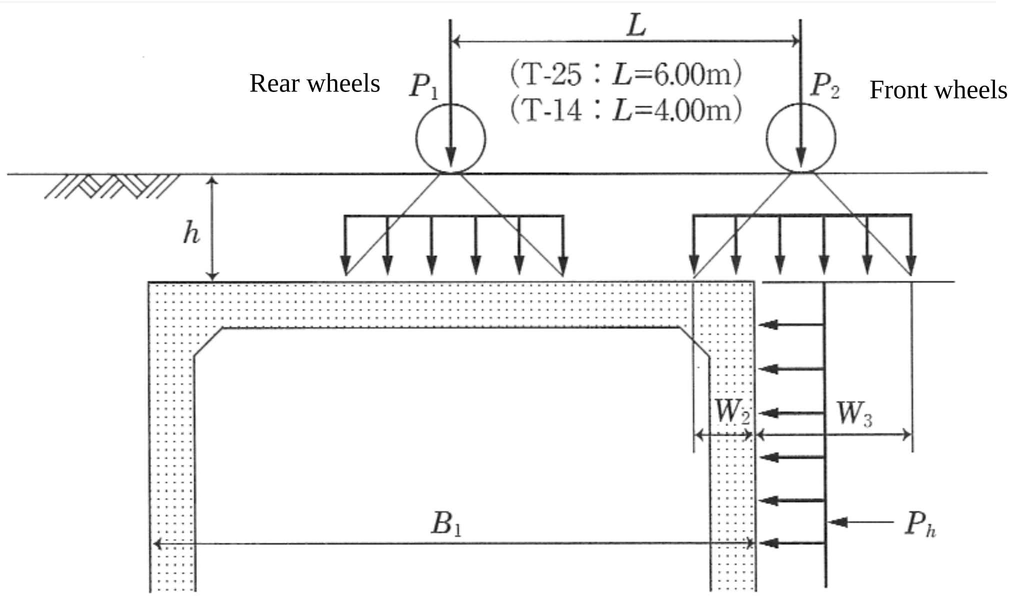


Fig. 3.13 Load distribution due to effect of front wheels

d) Box culvert top part used as road

When the longitudinal direction of box culvert and the vehicle traveling direction are the same, the rear wheel load with tread width of 0.2 m is considered a distributed load spreading downward at an angle of 45° in the longitudinal direction of box culvert (see Fig. 3.15). And for the span direction, different application shall be chosen for the following cases depending on the field conditions (see Fig. 3.14).

Case A: Full carriageway width (B2) ≥ Full width of box culvert (B1)
Wheel load shall be loaded over the full width of box culvert.

Case B: Full carriageway width (B2) < Full width of box culvert (B1) or, Administrative use road etc. (B3) < Full width of box culvert (B1)
Loading width shall be the loading width of wheel load and B2, or one suitable for the actual conditions.

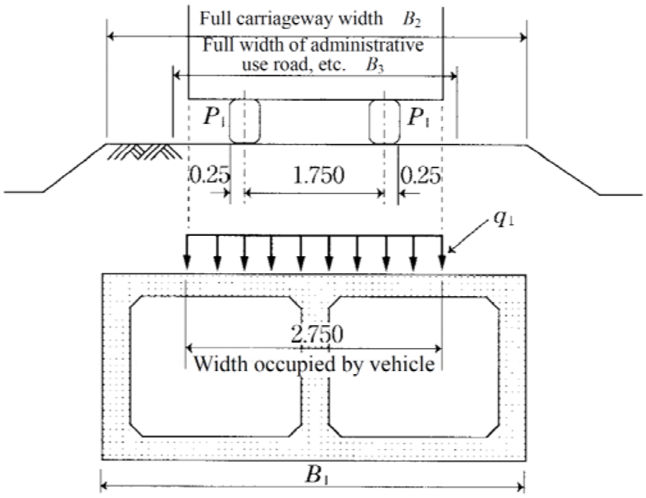


Fig. 3.14
Load distribution in span direction

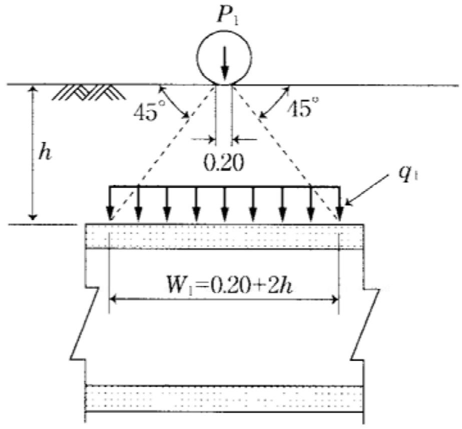


Fig. 3.15
Load distribution in longitudinal direction (road transverse direction)

Load strength per unit meter in longitudinal direction (kN/m)

$$P_w = \frac{2P_1(1+i)}{W_1} \qquad \text{................................ (F. 3.36)}$$

However, Fig. 3.14 describes the Case B.

$$q_1 = \frac{P_w \cdot \beta}{\text{Width occupied by vehicle}} \qquad \text{................(F. 3.37)}$$

Where P_w: Load strength due to wheel load (kN/m)

P_1: Rear wheel load (kN)

i: Impact coefficient (Table 3.12)

W_1: Load distribution width in longitudinal direction (m)

$= 0.2 + 2\,h$

However, the load distribution width W_1 shall be 1.0 m when the earth covering h is less than 0.4 m.

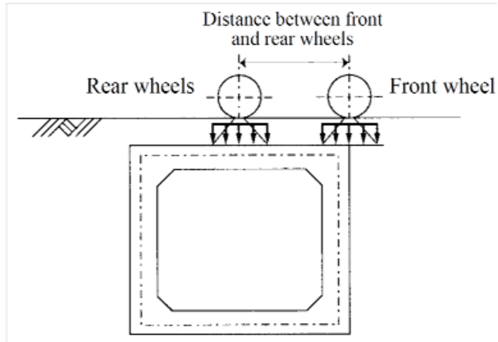
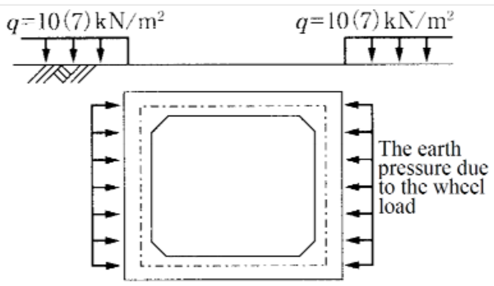
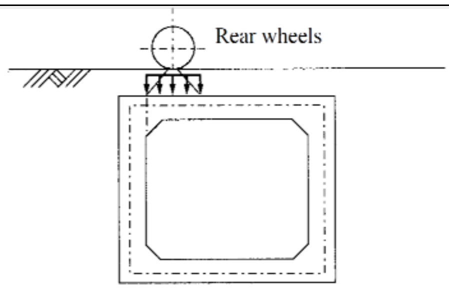
q_1: Load strength due to wheel load (kN/m^2)

β: Reduction coefficient of stress resultant (Table 3.13)

e) Loading position

The basic concept of loading position of vehicle wheel loads shall follow the principle that the position at which the maximum stress is generated in design analysis of each member shall be loaded. In the case of box culvert, design of section is designed under condition of the maximum member stress based on consideration of various load combinations described in Table 3.14.

Table 3.14 Loading position of single-line box culvert
(In the case of road crossing over box culvert)

Case	Loading condition	Loading diagram
1	• The real wheels are loaded on the span center. When the effect of front wheels needs to be considered (e.g. long span or deep earth covering), the front wheels shall be loaded as well. • The bending moment of top slab grows larger.	
2	• The horizontal earth pressure due to the wheel load (10 kN/m^2) shall be considered without loading the wheel load on the top slab. (In the case of T-25) Also, 7 kN/m^2 shall be used in the case of T-14. • The bending moment of sidewall grows larger.	
3	• When the wheel load is concentrated to some degree, or when the front wheels are loaded as well, the rear wheels shall be loaded on the edge of the top slab to study the shearing force in the top slab. • The shearing force in the top slab grows larger.	

(6) Crowd loads

The crowd load is considered for sidewalks, etc., and the magnitude of vertical load shall be according to the following description. For the farm road where no large-sized motor vehicle would travel, and for the sidewalk of public road (including road surfaces where large-sized motor vehicles may travel), the vertical load shall be 3 kN/m^2 and 5 kN/m^2, respectively. However, the vehicle load and the sidewalk dynamic load shall not be applied at the same time.

(7) Track loads

The vertical load due to track load consists of the standard dynamic load by line category and the overburden loads including rails, railroad ties, track bed, etc. Since the calculation of overburden loads is generally subjected to discussion with parties involved, it is necessary to consult with the road administrator to determine them.

(8) Earthquake load (reference)

In designing canal structures, earthquake loads must be considered for large size piers, retaining walls, aqueducts, canals with large banking and other important structures for earthquake proofing analysis, seismic coefficient method, modified seismic coefficient method, displacement method any dynamic analysis method are employed. Generally, the seismic coefficient method is used for the design of canal structures. However, when survey that is more detailed should be done, other methods may be employed depending on purposes. In case of the seismic coefficient method, the earthquake load is assumed by the dead load applied to the structure multiplied by the design seismic coefficient.

1) Design horizontal seismic coefficient

The design horizontal seismic coefficient is calculated by using the following formula (F. 3.38):

$$K_h = C_s \cdot K_0 \quad \cdots\cdots\cdots\cdots\cdots\cdots (F.\ 3.38)$$

Where K_h : Design horizontal seismic coefficient

K_0 : Standard design horizontal seismic coefficient (Usually 0.2)

C_s : Correction factor by importance of structure

Note: The design horizontal seismic coefficient should be more than 0.1.

Table 3.15 Correction factor by importance of structure (C_s)

Category	Correction factor
Structures of particular importance	1.0
Structures in general Gigantic rigid structures	0.7 - 0.8
Earth structure	0.5 - 0.7

2) Design vertical seismic coefficient

In principle, design vertical seismic coefficient is not considered for the earthquake load analysis. However, in the case of designing of support or bearing design vertical seismic coefficient may be required as value of 0.1.

(9) Wind pressure load

The wind pressure load must be considered for large scale piers, aqueducts and other facilities which are affected by wind. The wind pressure load is determined according to the design specification for road bridges from consideration of scale of structure, material of members and types, etc.

(10) Construction load

During the construction period, construction loads must be considered for the dead weight of structure and construction equipments.

(11) Temperature stresses

Temperature stresses must be considered for designing of statically indeterminate structures.

1) Scope of application of temperature in plain concrete structures may be determined with consideration of annual temperature data uniform 15° C is applied for design. In the case of structure whose height is less than 70 cm, 10° C is used instead of 15°C.

2) Liner expansion coefficient used for the design is as follows:

- The liner expansion coefficient of 12×10^{-6}/° C is taken for the design of steel structures.
- In the case of concrete structures, 10×10^{-6} /° C is taken as the liner expansion coefficient.

(12) Drying shrinkage and creep of concrete

In designing important structures, drying shrinkage and creep of concrete considering for statically indeterminate structure are as follows:

1) Drying shrinkages used for design analysis of statically indeterminate structure are show in Table 3.16.

2) In designing of plain concrete structures, generally drying shrinkage of 25×10^{-5} is taken in the normal case.

Table 3.16 Drying Shrinkage

Type of structure	Drying Shrinkage
Rigid frame	15×10^{-5}
Arch – steel ratio (> 0.5%) steel ratio (0.1%<. <0.5%)	15×10^{-5} 15×10^{-5}

3) Deformation of creep increases in proportion to elastic deformation.
Creep deformation of 3.0 is taken for indoor and 2.0 is applied for outdoor in general.

(13) Frost heave pressure
Frost heave pressure must be considered in cold areas. It is difficult to study on frost heave pressure theoretically; therefore, experimental frost heave prevention measures must be taken.

3.3.3 Reaction of Foundations

Reactions of base foundations shall be analyzed on the assumption that no deflection is generated in structures.

(1) When a uniform load acts on normal (compressive) ground;
The reaction of the foundation should, strictly speaking, be determined by analysis of the reaction and settlement (deflection) of beams or slabs placed on an elastic foundation. However, it has been shown that the calculated result is not much different from that of a uniformly distributed loading. This approach is therefore to be preferred.

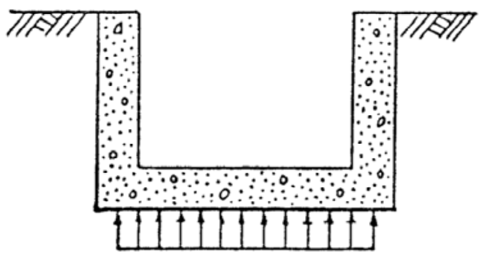


Fig. 3.16 Foundation reaction (Compressive foundation)

(2) In the case of an eccentric load
When the resultant force of the total load applied to the foundation ground is eccentric, the ground reaction shall be determined by the following

formula on the assumption that there is no effect from the ground compressibility.

1) When the resultant force is within the "middle third" of the center.

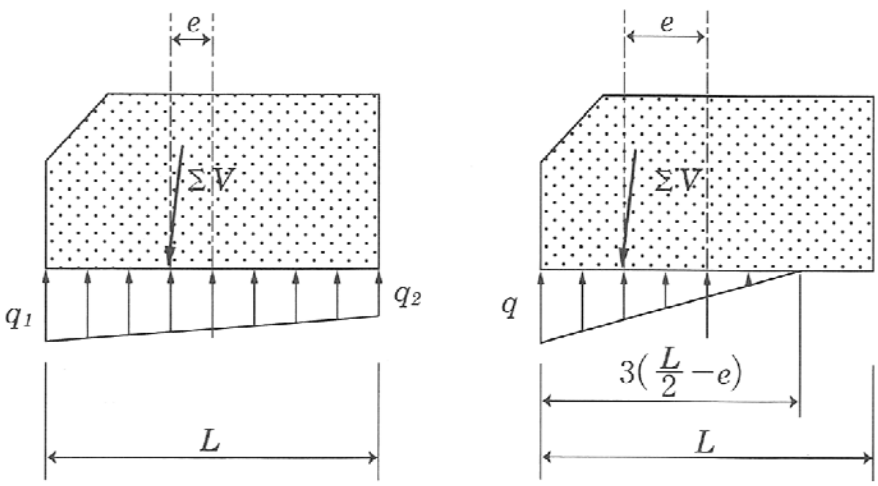
$$\left.\begin{matrix} q1 \\ q2 \end{matrix}\right\} = \frac{N}{L} \cdot \left(1 \pm \frac{6e}{L}\right) \text{……………..(F. 3.39)}$$

Where q_1, q_2 : Reaction strength at each and (t/m^2)

N : Vertical component of force of the resultant force (t/m)

L : Length of foundation

e : Eccentric distance of an acting point N



(Within the middle third) (Outside the middle third)

Fig. 3.17 Ground reaction with eccentric load

2) When an acting point of the resultant force is outside the "middle-third " of the center.

$$q = \frac{4}{3} \cdot \left(\frac{N}{L - 2e}\right) \text{……………..(F. 3.40)}$$

3.3.4 Loads Acting on Open Channels

Loads to be considered in structural design of the canal are, in general, dead weight, water pressure (internal and external), buoyancy/uplift pressure, earth pressure, vehicle load, impact load, crowd load and ground reaction.

(1) Load combination

Common combinations acting on canals built on common ground are described in Fig. 3.18 as well as in Table 3.17. Calculations shall be performed based on an appropriate load combination selected in consideration of the site conditions of the canal, construction condition, soil condition, etc.

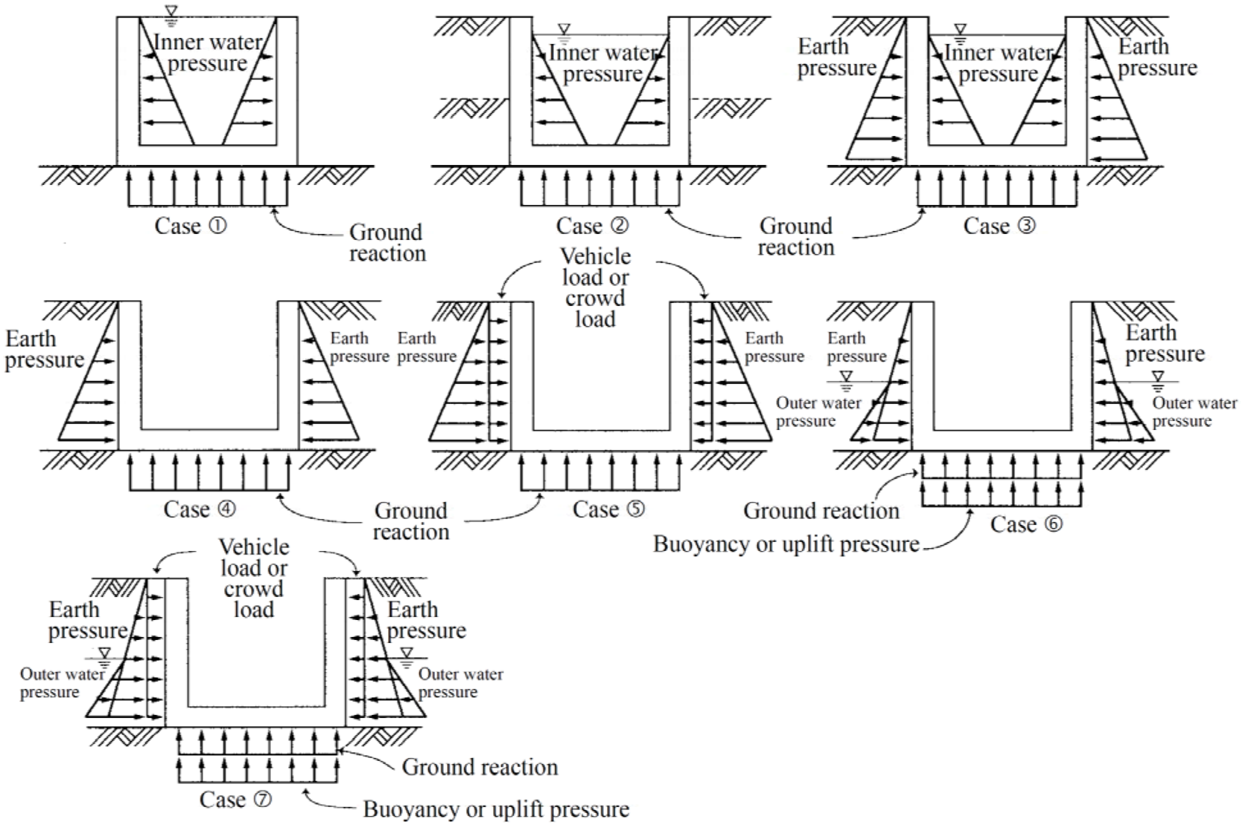


Fig. 3.18 Load combination applied to the canal

Table 3.17 Load combination applied to the canal

Case	Condition of wall	Internal load	External load	Remarks
①	Exposed	Inner water pressure under water level up to the wall top	Ground reaction	No backfill is provided
②[1)]	Backfilled	Inner water pressure under the expected highest water level	Ground reaction	No action by earth pressure due to backfill soil
③	Backfilled	Inner water pressure under the expected highest water level	Earth pressure due to backfill soil up to ground surface + Ground reaction	Action of earth pressure due to backfill soil is expected.
④	Backfilled		Earth pressure + Ground reaction	Groundwater, vehicle load, or crowd load are not expected.
⑤[2)]	Backfilled		Earth pressure + Embankment load + Vehicle load (or Crowd load) + Ground reaction	No groundwater is expected.
⑥	Backfilled		Earth pressure + Outer water pressure + Buoyancy (or Uplift pressure) + Ground reaction	Neither vehicle load nor crowd load is expected.
⑦[2)]	Backfilled		Earth pressure + Outer water pressure + Embankment load + Vehicle load (or Crowd load) + Buoyancy (or Uplift pressure) + Ground reaction	Groundwater and vehicle load (or crowd load) are expected.

Notes: 1. Case ② shall be applicable to situations when no acting earth pressure or sufficiently small acting earth pressure is expected. Examples include situations when freestanding is possible by the angle of repose or cohesion with a good backfill soil, or when the height of backfill is small. The embankment load shall be considered depending on circumstances.

2. The vehicle load and the sidewalk dynamic load can be applied, provided that they are not acting concurrently.

(2) Soil constant

For the earth pressure calculation, the soil constant such as soil unit weight, internal friction angle, cohesion and others should be necessary. Since these soil constants show different values according to the conditions of soil property, drainage, constructional method and others, investigation such as the soil test shall be carried out to obtain the proper value. However, it requires much labor and time to carry out these investigation and tests, and also requires sufficient experience and high technique to decide the proper soil constant. Therefore, the standard value shown in Table 3.18 can be used for the normal design, but not for the special cases such as a very important structure, a large scaled earthwork or soft cohesive soil.

Table 3.18 Soil unit weight

(Unit: kN/m^3)

	Type of soil	Saturated unit weight	Wet unit weight	Angle of shear resistance (°)
1	Gravel, coarse sand, etc., that hardly contain fine-grained soil. (Fine-grained fraction less than 5% such as GP, GW, SP, SW, etc., as a guideline)	20	18	30
2	Gravel, sand, etc., that contain fine-grained soil. (Fine-grained fraction 5 ~ 15% such as G-F, S-F, etc., as a guideline)	20	18	25
3	Silty fine sand, gravel that contains clay, etc. (Fine-grained fraction 15 ~ 50% such as GF, SF, etc., as a guideline)	20	18	20

(3) Load applied to vertical walls

1) Water pressure (internal water pressure and external water pressure)

The hydrostatic pressure p at the water depth h acting on internal and external surfaces of a vertical wall shall be obtained by $p = \gamma_w \cdot h$, which acts perpendicular to the wall surface. Note that γ_w is the unit weight of water.

2) Earth pressure

(a) Earth pressure classification and applicable structures

The earth pressure acting on the vertical wall of canal is classified as shown below, and the equation for each classification is provided in Table 3.20. Also, when the ground behind of the wall surface has irregular surface, the earth pressure shall be obtained by using the Trial Wedge Method, etc.

a) Structures in which only deformation is considered. (e.g. flume)

The earth pressure in this case is the value used in the structural calculation of the wall, and the wall friction shall be considered depending on circumstances. The active earth pressure acting on the wall surface varies with the slope condition of the ground surface, loading condition, etc. Although the angle of wall friction varies depending on the wall surface inclination, with or without the overhang, the values provided in Table 3.19 shall be used.

Additionally, when the outer water pressure is considered (Case-⑥ and Case-⑦ in Table 3.17), the submerged unit weight of soil shall be used for the unit weight of backfill soil at or below the groundwater level, and the coefficient of earth pressure and the angle of wall friction shall remain unchanged (see ⑥ of Table 3.20).

b) Structures in which movement and deformation are considered. (e.g. retaining wall)

The earth pressure in this case is the value used in both the structural calculation of wall and the stability analysis of structure, and the wall friction shall be considered. Although the angle of wall friction varies depending on the concrete placement condition, backfill condition, etc., the values provided in Table 3.19 shall be used as the standard. The earth pressure in this case can be obtained by using equations (F. 3.41) through (F. 3.48).

Table 3.19 Values for angle of wall friction (δ)

Types of retaining wall		Gravity type retaining wall Leaning type retaining wall Concrete block retaining wall	Inverted T-type retaining wall L-shaped retaining wall		Flume	
Types of analysis		Stability analysis	Stability analysis	Member analysis	Stability analysis	Member analysis
Types of friction angle		Soil and concrete	Soil and soil	Soil and concrete	Soil and soil	Soil and concrete
Angle of wall friction	Under normal condition	$\frac{2}{3}\phi$	β (Fig. 3.19)$^{1)}$	$\frac{2}{3}\phi$	$\phi^{3)}$	$\frac{2}{3}\phi$
	Under seismic condition	$\frac{1}{2}\phi$	$\tan\delta = \frac{\sin\phi \cdot \sin(\theta_0 + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta_0 + \Delta - \beta)}^{2)}$ However, that $\sin\Delta = \frac{\sin(\beta + \theta_0)}{\sin\phi}$	$\frac{1}{2}\phi$	$\frac{1}{2}\phi$	$\frac{1}{2}\phi$

Notes:
1. When $\beta \geq \phi$, then $\delta = \phi$.
2. When $\beta + \theta_0 \geq \phi$, then $\delta = \phi$.
   where θ_0: Seismic compound angle ($= \tan^{-1}\frac{K_h}{1-K_v}$)
3. The angle of wall friction (δ) shall be (2/3) ϕ according to the inclination of side wall and the overhung width of base.

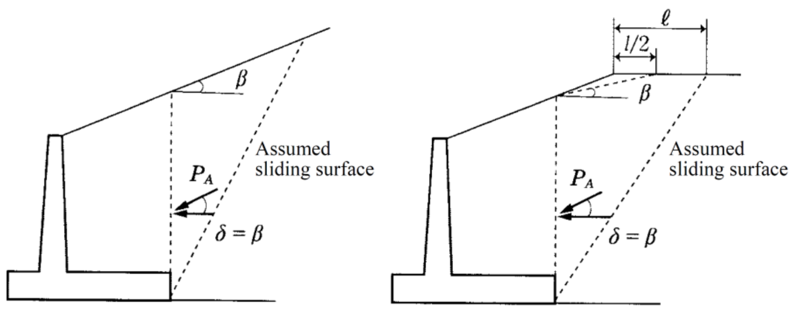


Fig. 3.19 Method to establish β

Table 3.20 Classification of earth pressure acting on canal wall (1/3)

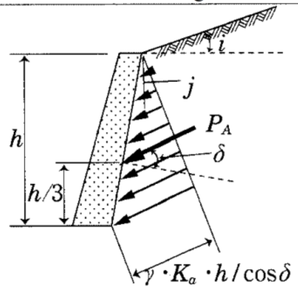
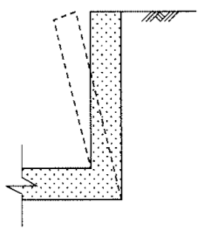
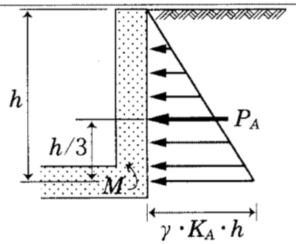
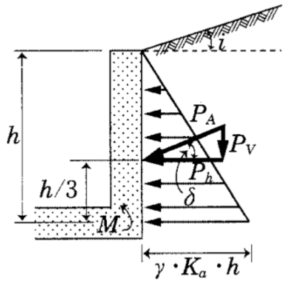
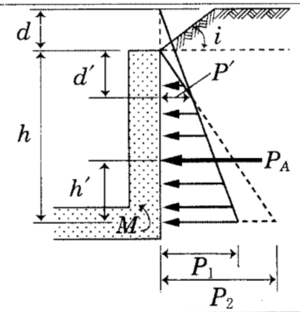
Applicable condition		Number	Illustrative diagram	Equation	Applicable classification	Remarks
	Basic equation	①		$P_A=\frac{1}{2}\gamma\cdot h^2\frac{K_a}{\cos j\cdot\cos\delta}$ $K_a=\frac{\cos^2(\phi+j)\cos\delta}{\cos(\delta-j)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\ \sin(\phi-i)}{\cos(i+j)\ \cos(\delta-j)}}\right\}^2}$	$\delta=0$ for cases ②, ④, ⑦, ⑧, ⑪, ⑫. $\delta=i$ for cases ⑨	The equation is the Coulomb's earth pressure formula. K_A (K_a): Coefficient of active earth pressure P_A: Active earth pressure ϕ: Angle of shear resistance of soil δ: Friction angle between soil and wall γ: Wet unit weight
Only the deformation is considered. 	When stability analysis is not accompanied.	②		$P_A=\frac{1}{2}\gamma\cdot h^2\cdot K_A$ $M=\frac{1}{6}\gamma\cdot h^3\cdot K_A$ $K_A=\frac{1-\sin\phi}{1+\sin\phi}$	Applicable in case of vertical wall and horizontal ground surface Applicable to case where the friction angle between soil and wall is to be ignored	The equation below shall be used when the cohesion c is considered. $K_A=\tan^2(45^\circ-\frac{\phi}{2})-\frac{2c}{\gamma\cdot h}\tan(45^\circ-\frac{\phi}{2})$
		③		$P_A=\frac{1}{2}\gamma\cdot h^2\cdot\frac{K_a}{\cos\delta}$ $P_h=P_A\cdot\cos\delta$、$P_V=P_A\cdot\sin\delta$、$M_h=P_h\cdot\frac{h}{3}$ $K_a=\frac{\cos^2\phi}{\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-i)}{\cos i\cdot\cos\delta}}\right\}^2}$	Applicable in case of vertical wall and sloped ground surface Applicable to case where the friction angle between soil and wall is to be considered.	$i\le\phi$, and so forth.
		④		$d'=\frac{K_{A1}\cdot p'}{K_{A2}-K_{A1}}$、$P'=\frac{K_{A1}\cdot K_{A2}\cdot\gamma\cdot d}{K_{A2}-K_{A1}}$ $P_1=K_{A1}(h+d)\gamma$ $P_2=K_{A2}\cdot h\cdot\gamma$ $P_A=\frac{1}{2}d'p'+\frac{h-d'}{2}(P'+P_1)$	Applicable in case of vertical wall and to case where ground surface has become horizontal as result of inclination correction	Method which combines the case of surcharge and the case of slope ground surface. Case ⑤ is applicable.

Table 3.20 Classification of earth pressure acting on canal wall (2/3)

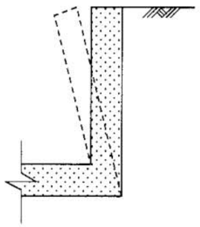
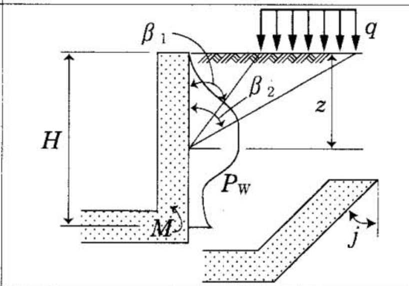
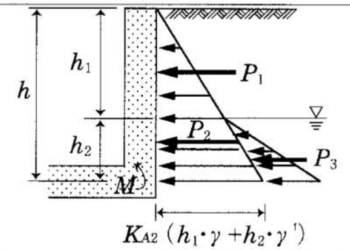
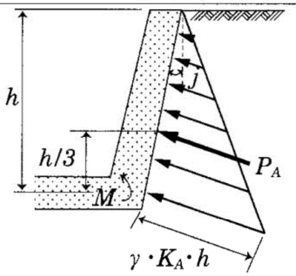
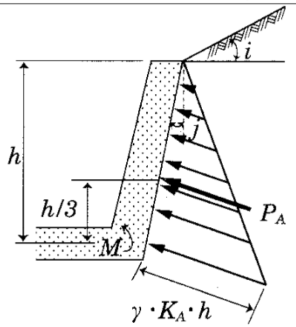
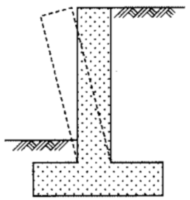
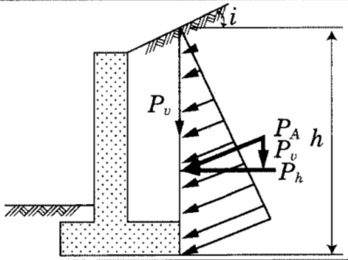
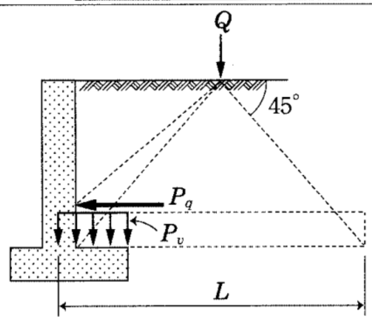
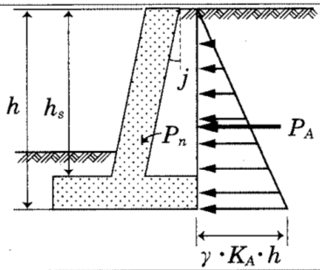
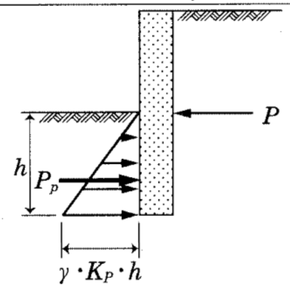
Applicable condition		Number	Illustrative diagram	Equation	Applicable classification	Remarks
Only the deformation is considered. 	When stability analysis is not accompanied.	⑤		$\sigma_z=\frac{q}{2}(2\varepsilon+\sin 2\varepsilon\cdot\cos 2\theta)\cdot K_A$ $\varepsilon=(\beta_2-\beta_1)/2$、$\theta=(\beta_1+\beta_2)/2$ $P_q=\int_0^H\sigma_z\cdot d_z$、$M=\int_0^H\sigma_z(H-z)\,d_z$ σ_z: Horizontal load strength applied on wall surface at depth z from the ground surface due to surcharge q P_q: Total horizontal earth pressure applied on wall surface due to surcharge q M: Bending moment at the bottom of wall due to surcharge q	Applicable to case where the surcharge is applied at some distance away from the edge of canal wall. (Embankment load, vehicle load, and sidewalk dynamic load)	The equation is the Freulich's formula. See to the Section "7.3.3 Conversion of surcharge" when the calculation is performed.
		⑥		$P_A=P_1+P_2+P_3$、$P_1=\frac{1}{2}\gamma\cdot h_1{}^2\cdot K_{A1}$ $P_2=\frac{h_2}{2}\{K_{A1}\cdot h_1\cdot\gamma+K_{A2}(h_1\cdot\gamma+h_2\cdot\gamma')\}$ $P_3=\frac{1}{2}\gamma_w\cdot h_2{}^2$	Applicable to case where the groundwater level is below the drainage hole. The coefficient of earth pressure shall remain constant regardless of the groundwater level.	γ': Unit weight of submerged soil $=\gamma_s-\gamma_w$ γ_s: Unit weight of saturated soil γ_w: Unit weight of water See case ① or case ③ when the friction angle between soil and wall is considered.
		⑦		$P_A=\frac{1}{2}\gamma\cdot h^2\frac{K_A}{\cos j}$ $K_A=\frac{\cos^2(\phi+j)}{\cos^2 j\ \left(1-\frac{\sin\phi}{\cos j}\right)^2}$	Applicable in case of inclined wall and horizontal ground surface	Treat in a manner similar to case ⑥ when there is groundwater.
		⑧		$P_A=\frac{1}{2}\gamma\cdot h^2\frac{K_A}{\cos j}$ $K_A=\frac{\cos^2(\phi+j)}{\cos^2 j\ \left\{1+\sqrt{\frac{\sin\phi\cdot\sin(\phi-i)}{\cos(j+i)\ \cos j}}\right\}^2}$	Applicable in case of inclined wall and sloping ground surface	Treat in a manner similar to case ④ as a composite method when the sloped ground surface becomes horizontal at some point.

Table 3.20 Classification of earth pressure acting on canal wall (3/3)

Applicable condition		Number	Illustrative diagram	Equation	Applicable classification	Remarks
Only the movement and deformation are considered. 	When stability analysis is accompanied.	⑨		$P_A=\frac{1}{2}\gamma\cdot h^2\cdot K_A$ $P_h=P_A\cdot\cos i$、$P_v=P_A\cdot\sin i$ $K_A=\left(\frac{\cos\phi}{\cos i+\sqrt{\sin^2\phi-\sin^2 i}}\right)^2$	Applicable to the inverted T-type retaining wall with vertical wall and all gravity retaining walls.	$\delta=i$ Assign zero to i in the equation for level ground surface.
		⑩		For P_q, apply case-⑤ by converting Q into the distributed load. $P_v=\frac{Q}{L}$	Load transfer to wall and bottom slab when the retaining wall is subject to a concentrated load.	
		⑪		$P_A=\frac{1}{2}\gamma\cdot h^2\cdot K_A$、$K_A=\frac{1-\sin\phi}{1+\sin\phi}$ $P_n=\frac{1}{2}\gamma\cdot h_s^2\cdot\frac{K_n}{\cos j}$ $K_n=\frac{\cos^2(\phi+j)}{\cos^2 j\left(1+\frac{\sin\phi}{\cos j}\right)^2}$	Applicable to the inverted T-type retaining wall with inclined wall.	P_A and P_n are used for the stability analysis and structural calculation, respectively.
	Passive earth pressure	⑫		$P_P=\frac{1}{2}\gamma\cdot h^2\cdot K_P$、$K_P=\frac{1}{K_A}=\frac{1+\sin\phi}{1-\sin\phi}$	Applicable to the passive earth pressure on vertical wall	For all situations including inclined walls and others. $K_P=\frac{1}{K_A}$

(b) Calculation of earth pressure by using the Trial Wedge Method

In the Trial Wedge Method, which is also called the extreme value method, the inclination of sliding surface that is generated at the bottom of wall is varied in various ways as shown in Fig. 3.20. To establish a balance of forces acting on the wedge shape portion between each sliding surface and the wall surface, using graphical methods or numerical calculations. From this result, the local maximum value of earth pressure acting on the wall, which is then treated as the active earth pressure, is obtained.

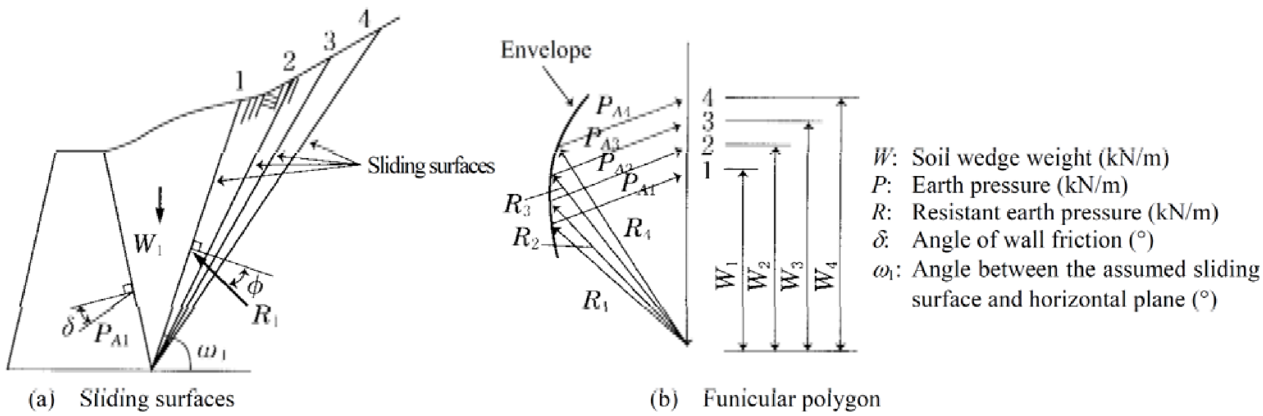


Fig. 3.20 Method to determine active sliding surface and active earth pressure

Culumann's graphical solution based on earth pressure theory of the Coulomb's series is one of common methods to obtain the earth pressure by graphics. Culumann's graphical solution can be described as follows. (Fig. 3.21)

(i) From the point A, draw a straight line having an angle of φ with the horizontal plane.

(ii) For arbitrary assumed sliding surfaces (A-1, A-2, and so on), weight of the block framed by ABCD1, ABCD2, and so on for each sliding surface (W1, W2, and so on, respectively) is calculated, and plot the results on the A-G line with appropriate scale.

(iii) From each of the plotted points (W1, W2, and so on), draw a straight line having an angle of ε with the line A-G. The point of intersection of each line and corresponding sliding surface

(A-1, A-2, and so on, respectively) is denoted by H, J, and so on, respectively. Plot a curve line to connect these points of intersection.

(iv) Lastly, draw a tangent on the curve connecting A, H, J, and so on, in a direction parallel to A-G. When the contact point is denoted by T, the magnitude of resultant of active earth pressure is defined by the line T-W, and the sliding surface is defined by the line A-T.

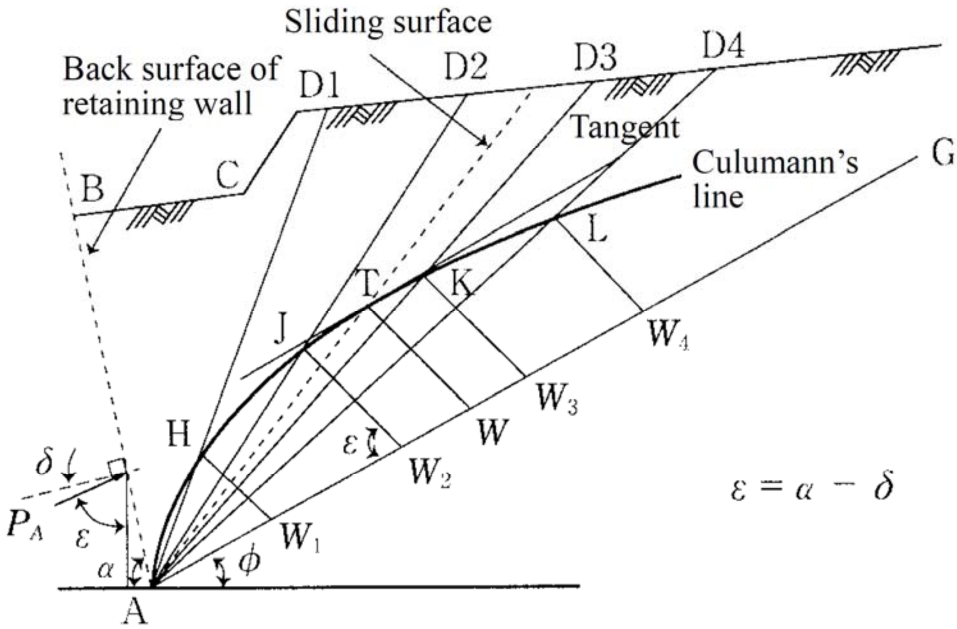


Fig. 3.21 Culumann's graphical method

a) Active earth pressure under normal condition (Fig. 3.23)

The ultimate condition in which soil wedge is about to slide on the sliding surface along the retaining wall is called the active state, and the earth pressure PA at this moment is called the active earth pressure. The three forces of the weight W of wedge shaped clod and the reaction forces of sliding surface PA and R are balanced, and the relationship of these forces can be illustrated in a funicular polygon as shown in Fig. 3.23 (b). And the active earth pressure PA can be obtained in the equation (F. 3.41) by using the sine formula.

$$\frac{P_A}{\sin(\omega-\phi)} = \frac{W}{\sin\{90^\circ-(\omega-\phi-\delta-\alpha)}$$

$$\therefore P_A = \frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)} W \quad \text{...(F. 3.41)}$$

where P_A: Active earth pressure (kN/m)
W: Weight of soil wedge (including overburden load) (kN/m)
ϕ: Angle of shear resistance of backside ground (°)
α: Angle between wall's back surface and vertical plane (°) (Fig. 3.22)
δ: Angle of wall friction (°) (Table 3.19)
ω: Angle between the assumed sliding surface and horizontal plane (°)

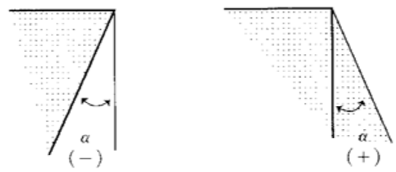
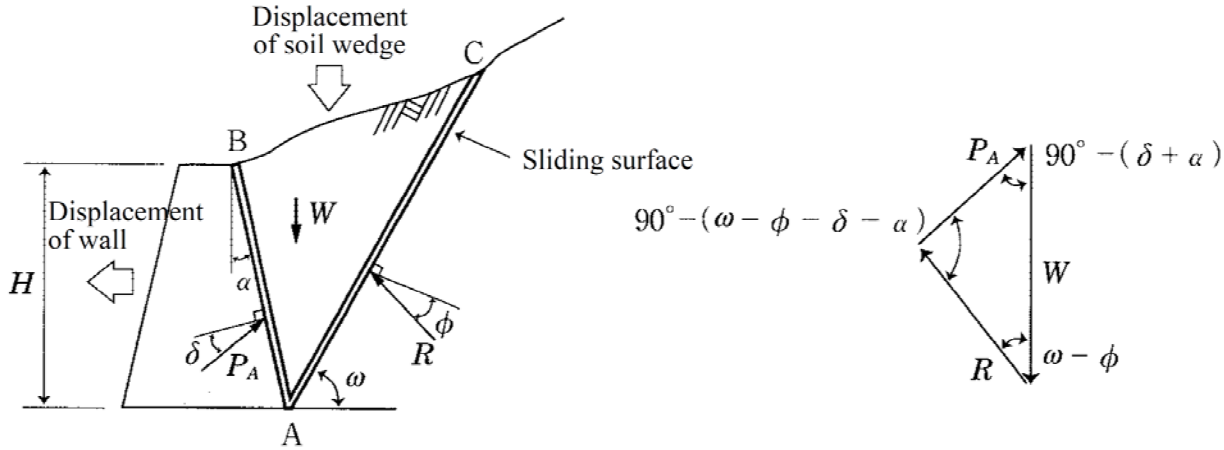


Fig. 3.22 Illustrated examples of positive and negative values for α



(a) Sliding surface (b) Funicular polygon

Fig. 3.23 Illustrative graphics of active earth pressure

Also, when P_A is decomposed into horizontal and vertical components, the equation (F. 3.42) is obtained, and the height of the action point of earth pressure is obtained by using the equation (F. 3.43).

$$\left.\begin{aligned} P_{AH} &= P_A \cos(\delta + \alpha) \\ P_{AV} &= P_A \sin(\delta + \alpha) \end{aligned}\right\} \quad \text{.......................(F. 3.42)}$$

$$hp = 1/3 * H \quad \text{.......................(F. 3.43)}$$

Where P_{AH}: Horizontal component of active earth pressure (kN/m)
P_{AV}: Vertical component of active earth pressure (kN/m)
h_p: Vertical distance to the action point of active earth pressure (m)
H: Height of wall (m)

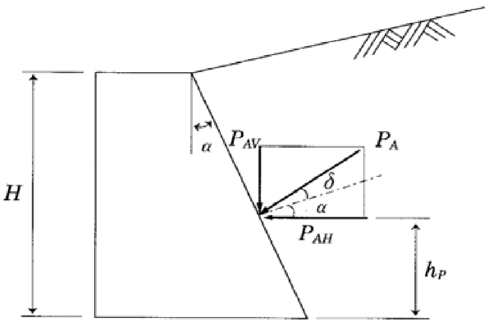
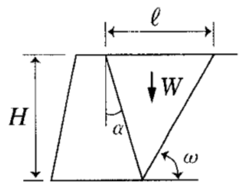
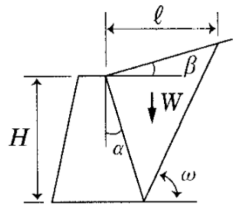
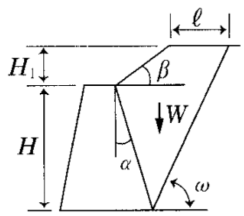
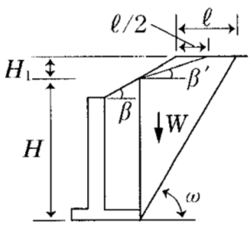
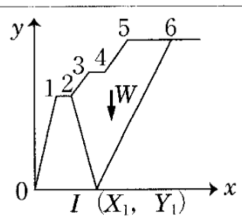


Fig. 3.24 Horizontal and vertical components of active earth pressure under normal condition

Table 3.21 provides formulas for soil wedge weights of typical backfill forms.

Table 3.21 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot H$
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha) \cdot \cos(\alpha-\beta)}{\cos^2\alpha \cdot \sin(\omega-\beta)}$ $\ell = \frac{\cos\beta \cdot \cos(\omega-\alpha)}{\cos\alpha \cdot \sin(\omega-\beta)} \cdot H$
	$W = \frac{1}{2}\gamma \left\{ (H+H_1)^2 \frac{\cos(\omega-\alpha)}{\sin\omega} - H_1^2 \frac{\cos(\alpha-\beta)}{\sin\beta} \right\} \frac{1}{\cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot (H+H_1) - \frac{\cos(\beta-\alpha)}{\sin\beta \cdot \cos\alpha} \cdot H_1$
	$W = \frac{1}{2}\gamma \{(H+H_1)^2 \cot\omega - H_1^2 \cot\beta\}$ $\beta' = \tan^{-1} \frac{2H_1}{(H+H_1)\cot\omega + H_1 \cot\beta}$ $\ell = (H+H_1)\cot\omega - H_1 \cot\beta$
	$W = \frac{1}{2}\gamma \sum_{i=1}^{n} (X_{i+1} - X_i)(Y_{i+1} + Y_1)$

b) Passive earth pressure under normal condition (Fig. 3.25)

The ultimate condition immediately before the backfill behind is ejected upward is called the passive earth pressure state, and the earth pressure "PP" at this point is called the passive earth pressure.

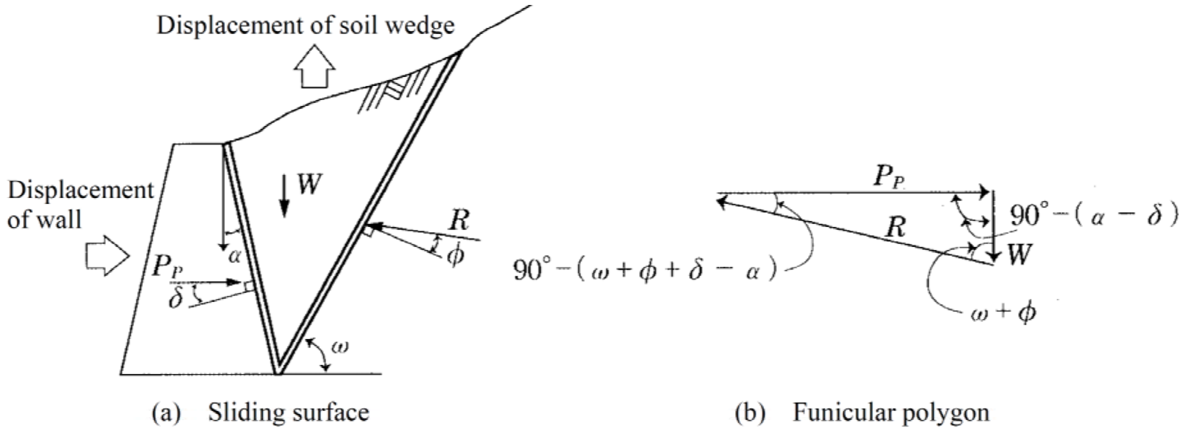


Fig. 3.25 Illustrative graphics of passive earth pressure

The passive earth pressure is obtained by using the equation (F. 3.44)

$$P_P = \frac{\sin(\omega+\phi)}{\cos(\omega+\phi+\delta-\alpha)} W \qquad \text{(F. 3.44)}$$

c) Active earth pressure under seismic condition

Under seismic condition, it is assumed that the inertia force (= Kh • W) acts on the center of gravity as shown in Fig. 3.27 (a). The funicular polygon at this moment can be illustrated as shown in Fig. 3.27 (b).

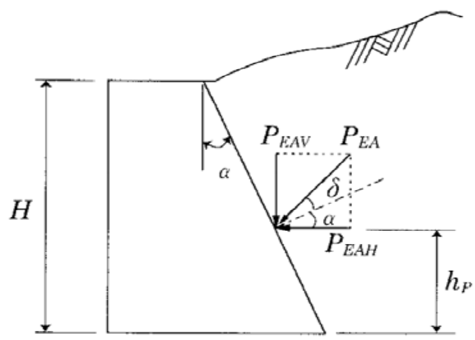
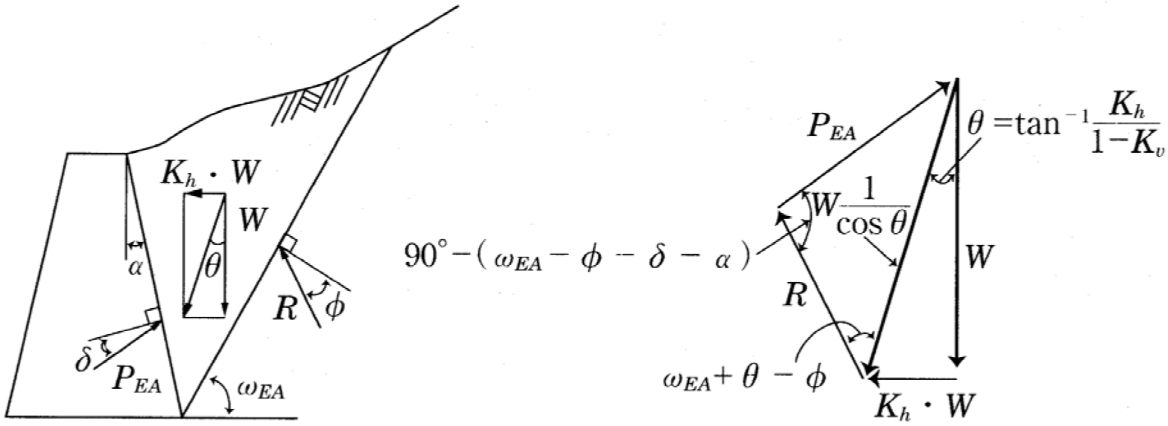


Fig. 3.26 Horizontal and vertical components of active earth pressure under seismic condition



(a) Sliding surface (b) Funicular polygon

Fig. 3.27 Concept of active earth pressure under seismic condition

$$P_{EA} = \frac{\sin(\omega_{EA}-\phi+\theta)}{\cos(\omega_{EA}-\phi-\delta-\alpha)\cdot\cos\theta}W \quad \text{(F. 3.45)}$$

$$\left.\begin{aligned} P_{EAH} &= P_{EA}\cos(\delta+\alpha) \\ P_{EAV} &= P_{EA}\sin(\delta+\alpha) \end{aligned}\right\} \quad \text{(F. 3.46)}$$

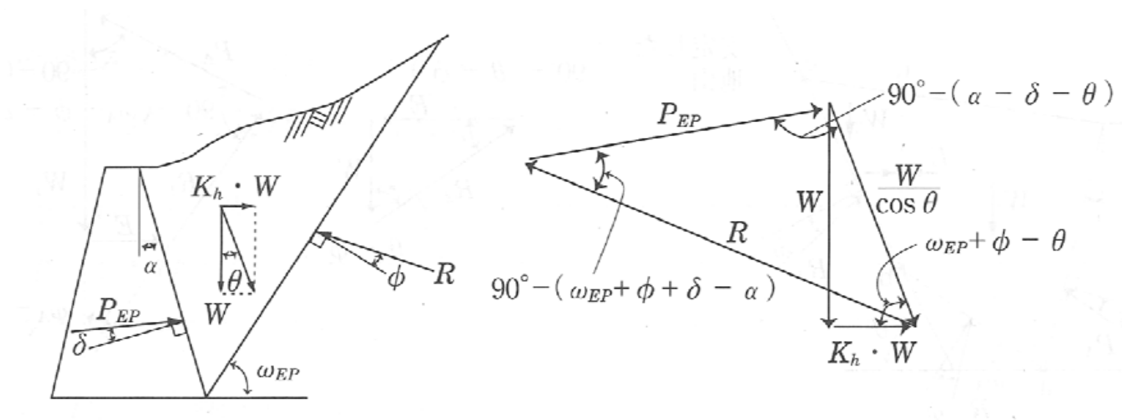
$$h_p = \frac{1}{3}H \quad \text{(F. 3.47)}$$

where P_{EA}: Active earth pressure under seismic condition (kN/m)
P_{EAH}: Horizontal component of active earth pressure under seismic condition (kN/m)
P_{EAV}: Vertical component of active earth pressure under seismic condition (kN/m)
ω_{EA}: Angle between assumed sliding surface and horizontal plane (°)
θ: Seismic compound angle ($\tan^{-1}\frac{K_h}{1-K_v}$) (°)
K_h: Design horizontal seismic coefficient (see the equation (F. 3.38))
δ: Angle of wall friction under seismic condition (°) (see the Table 3.19)
h_p: Vertical distance to the action point of active earth pressure under seismic condition (m)

d) Passive earth pressure under seismic condition (see Fig. 3.28)

$$P_{EP} = \frac{\sin(\omega_{EP}+\phi-\theta)}{\cos(\omega_{EP}+\phi+\delta-\alpha)\cos\theta}W \quad \text{(F.3.48)}$$

Where P_{EP}: Passive earth pressure under seismic condition (kN/m)
ω_{EPA}: Angle between assumed sliding surface and horizontal plane (°)



(a) Sliding surface (b) Funicular polygon

Fig. 3.28 Concept of passive earth pressure under seismic condition

[Reference] Earth pressure acting on the retaining wall in cut area

The retaining wall in cut area is a type of retaining wall to which the cut slope or structures such as existing retaining wall, etc., are located closely, thus the earth pressure is affected by such cut slope or structures. In the case of the retaining wall in cut area as shown in Fig. 3.29, since it is assumed that the sliding surface would be generated either inside of backfill soil (a to b) or on the plane interfacing with the cut slope surface (a to c), it is necessary to select the earth pressure of whichever is greater. While the earth pressure under the assumption of sliding surface generated inside the backfill soil can be obtained in the same manner as the retaining wall in fill area, it is necessary to use the angle of shear resistance (φ') of the interfacing plane when the sliding surface is assumed on the interfacing plane. While the angle of shear resistance of the interfacing plane varies depending on factors including the soil property of natural ground, roughness coefficient of the cut slope, etc. It is expected to be φ' = (1 to 2/3) φ (φ: angle of shear resistance of backfill soil) under normal condition. φ' = 2/3 φ when the soil is soft rock or better and the surface is relatively uniform plane, and φ' can be assumed to be equal to φ when the soil is coarse or bench cut is provided. Since the value of φ' affects the magnitude of earth pressure, it shall be determined with cautions. Additionally, the consideration for this earth pressure in cut area is applicable to situations where the gradient of natural ground is 45° or greater and the horizontal distance between the retaining wall's back surface and the bottom of slope is 1 m or less. Otherwise, there is no need to consider.

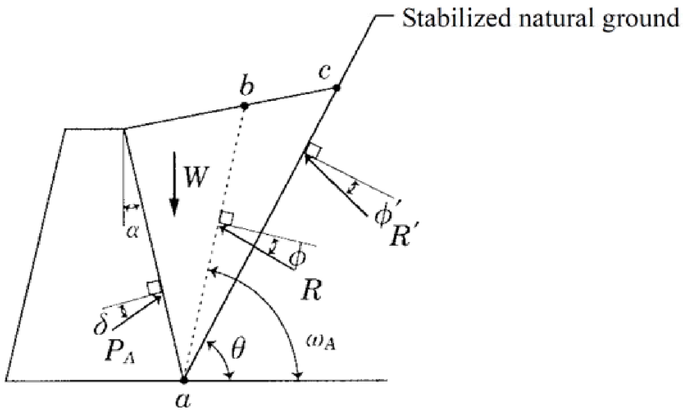


Fig. 3.29 Calculation method of earth pressure in cut area (part 1)

Also, when the retaining wall is apart from the cut slope as shown in Fig. 3.30, it is necessary to calculate the earth pressure based on the assumption that takes the segmented sliding surface through 3 given points a, b, c shown in the figure into account in addition to the sliding surface passing through the inside of backfill soil. The earth pressure in this case can be obtained as follows. At first, assume a sliding surface passing through the heel 'a', and then find the point b where the assumed sliding surface intersects with the cut slope. Draw a plumb line from the point b, and obtain 'b'' where the plumb line intersects with the ground surface. If it is assumed that the backfill soil divided along the line b-b' as shown in the Fig. 3.30 is equilibrated by internal force E, then the earth pressure acting on the retaining wall is obtained by using the equation (F. 3.49).

$$P_A = \frac{W_1 \sin(\omega_A - \phi) + W_2 \cos(\omega_A - \phi)\tan(\theta - \phi')}{\cos(\omega_A - \phi - \delta - \alpha)} \quad \text{......................(F.3.49)}$$

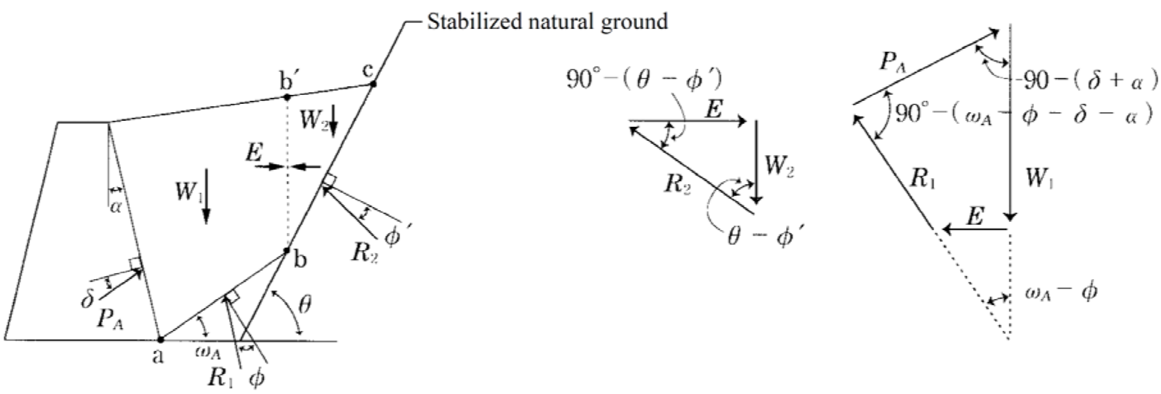


Fig. 3.30 Calculation method of earth pressure in cut area (2)

3.3.5 Stability Analysis

In designing a masonry canal etc, the load to be applied shall be determined in accordance with "3.3.4 Load applied to the Canal" to review each part of the canal structure. The review contents specifically include reviews of ① overturning, ② sliding, and ③ bearing capacity. Additionally, in the cases of soft ground area or sloping bearing layer, and in relevant situations, the stability of the ground as a whole shall be reviewed as necessary.

(1) Reviews for overturning (Fig. 3.31)
Conditions to stabilize structures against overturning shall satisfy the following values.

1) If $L / 2 < e$, then the structure overturns.
2) If $L / 6 < e \leq L / 2$, then the structure would not overturn, however, tensile stress is generated in the base surface part of structure. Therefore, conditions to stabilize structures against overturning shall be according to the equations (F. 3.50) and (F. 3.51).

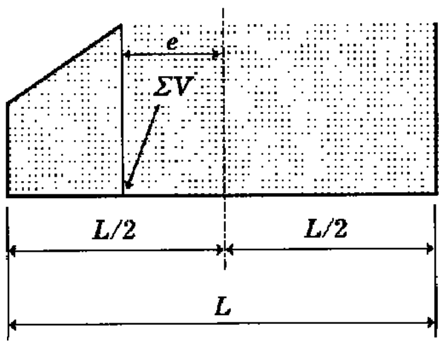


Fig. 3.31 Resultant action line

During normal condition $e \leq L / 6$ ....(F. 3.50)

During earthquake $e \leq L / 6$ ....(F. 3.51)

$e = L / 2 - X_0 = L / 2 - (\Sigma M_r - \Sigma M_0) / \Sigma V$

Where e: Distance between the center point of base surface and the point on which the resultant action line intersects with the base surface (m)
X_0: Action point of resultant (m)
ΣV: Total vertical load (kN/m)
ΣM_r: Total resisting moment at the origin (kN-m)
ΣM_0: Total overturning moment at the origin (kN-m)
L: Length of base surface (m)

(2) Reviews for sliding

The sliding resistance at foundation base surface directly against the total horizontal loads acting on the foundation base surface shall have the required safety factor. Consideration for sliding shall be in accordance with the equation (F. 3.52).

$R_H \; / \; \Sigma H \geq F_s$ ....(F. 3.52)

$R_H = \Sigma V \times f$ ....(F. 3.53)

Where R_H: Sliding resistance force (kN/m)
ΣH: Total horizontal load (kN/m)
ΣV: Total vertical load (kN/m)
When buoyancy exists due to the inside water level of canal, the horizontal and vertical loads should take this into consideration.
F_s: Safety factor (1.5 for normal condition and 1.2 when earthquake is considered.)
f : Coefficient of friction between base surface and foundation ground (Table 3.22)

Table 3.22 Coefficient of friction between base surface and foundation ground

Material	f
Earth and concrete (The side of cast-in-place concrete surface in which forms are not used)	tanØ
Earth and concrete (Surfaces other than described above)	Tan(2/3Ø)
Rock and concrete	Approximately 0.70

Note: φ denotes angle of shear resistance of the foundation ground soil.

Additionally, when the safety factor for sliding is insufficient, the structure with increased sliding resistance should be planned by providing protrusions, etc. in order to secure the required safety factor.

(3) Reviews for foundation ground bearing capacity

While there are various methods proposed to calculate the bearing capacity of ground for shallow foundations, the values listed in Table 3.23 are used for the purpose of study in this technical guideline.

Table 3.23 Allowable bearing capacity of foundation ground

Ground		Allowable bearing unit stress (kN/m^2)	Remarks	
			N-value	q_u (kN/m^2) *
Rock		1,000	100 and more	
Sand table		500	50 and more	
Hardpan layer		300	30 and more	
Gravel layer	Compacted Not compacted	600 300		
Sandy ground	Dense Medium Loose Very loose	300 { 200 100 50 0	30 - 50 { 20 - 30 10 - 20 5 - 10 5 or less	
Cray	Very hard Hard Medium Soft Very soft	200 100 50 20 0	15 - 30 8 - 15 4 - 8 2 - 4 0 - 2	250 and more 100 - 250 50 - 100 25 - 50 25 or less
Kanto loam	Hard Hardish Soft	150 100 50	5 and more 3 - 5 3 or less	150 and more 100 - 150 100 or less

* Unconfined compressive strength

According to “3.3.3 Reaction of Foundations”, consideration for foundation ground bearing capacity shall be in accordance with the equation (F. 3.54).

$$\left.\begin{matrix} q1 \\ q2 \end{matrix}\right\} = \frac{N}{L} \cdot \left(1 \pm \frac{6e}{L}\right) < qa \text{ ....(F. 3.54)}$$

Where q_1, q_2 : Reaction strength at each end (KN/m^2)

N : Vertical component of force of the resultant force (KN/m)

L : Length of foundation

e : Eccentric distance of an acting point N

qa : Allowable bearing capacity of foundation ground(KN/m^2)

3.3.6 Plain and Reinforced Concrete

(1) Types of reinforcing bars

Deformed reinforcing bars SD 295 A or B, and SD 345 are commonly used in canals or canal structures. Presently, with the exception of reinforced concrete secondary products, the use of round bars is on a declining trend.

1) Deformed reinforcing bar has large bond strength compared to round bar, and has strong resistance to slip against pull-out force. Additionally, it has a

function to disperse cracking generated in concrete. Also, splicing and anchorage of reinforcing bars are simplified with deformed reinforcing bars.
2) When the high-tensile-strength reinforcing bar (SD 345) is used for canal structures, it is necessary to verify that safety is sufficiently secured for crack width, deflection, etc. With the exception of particularly large-scale structures, SD 295 A is frequently used for common type canal structures.

(2) Allowable stresses

The 28-day strength specified for concrete shall be determined case by case in consideration of the scale of the structure, importance (durability and water-tightness), construction conditions (Plants and materials available), economic advantages, and other factors. Allowable stresses for reinforcement shall be determined based on the types of reinforcing bars used, strength of the concrete , and other factors, but for siphons and other structures subject to internal water pressure, the allowable stress of the reinforcing bars must be reduced to minimize cracking of concrete and thereby keep high water tightness.

1) Design strengths and allowable stress intensities for concrete

The 28-day design strength and allowable stress of the concrete commonly used in canal linings and canal structures are as described in Table 3.24 and Table 3.25. In addition, application categories for typical design strength are described in Table 3.26.

Table 3.24 Allowable stress of plain concrete (Unit: N/mm2)

28-day design strength σ_{ck} / Allowable stress	18	21	24	Remarks
Compressive stress (σ_{ca})	4.5	5.0	5.4	$\sigma_{ca} \leq \frac{\sigma_{ck}}{4}$
Tensile stress due to bending (σ_{ta})	0.25	0.29	0.29	$\sigma_{ta} \leq \frac{\sigma_{tk}}{7}$
Bearing stress (σ'_{ca})	5.4	5.9	5.9	$\sigma'_{ca} \leq 0.3\sigma_{ck}$

Notes: 1. σ_{ck} is the design strength of concrete.
2. σ_{tk} is the design tensile strength of concrete (specified by JIS A1113)

Table 3.25 Allowable stress of reinforced concrete (Unit: N/mm^2)

Allowable stress \ 28-day design strength σ_{ck}			18	21	24	30	40 or larger	Remarks
Compressive stress due to bending σ_{ca}			7	8	9	11	14	
Shear	Calculation of the diagonal tension bar is not performed. τ_{a1}	Beam	0.4	0.42	0.45	0.5	0.55	
		Slab[1)]	0.8	0.85	0.9	1.0	1.1	
	Calculation of the diagonal tension bar is performed. τ_{a2}	Shearing force only[2)]	1.8	1.9	2.0	2.2	2.4	
Bonding	Round bar τ_{0a1}		0.7	0.75	0.8	0.9	1.0	
	Deformed reinforcing bar τ_{0a2}		1.4	1.5	1.6	1.8	2.0	
Bearing stress σ'_{ca}			5.4	6.3	7.2	9.0	12.0	$\sigma'_{ca} \leq 0.3\sigma_{ck}$

Notes: 1. This is the value for punching shear.
2. This value may be increased when the effect of torsion is considered.

Table 3.26 Application categories of cast-in-place concrete (Unit: N/mm^2)

Design strength σ_{ck}	Items for application
Plain concrete $\sigma_{ck} = 18$	Leveling for the foundation of structures (leveling concrete) Plain concrete structures such as foundation concrete
Reinforced concrete $\sigma_{ck} = 21$	Canal structures such as flume, culvert, siphons, etc.
Reinforced concrete $\sigma_{ck} = 24$	This is applicable to case where durability is required (e.g. structures such as chute which overcomes wear due to water flow), or where the combined use of SD345 provides higher stability and economic efficiency of structures.

Note; This table provides application categories of typical design strengths, and the mix proportioning strength can be changed depending on field conditions and construction conditions. In addition, when enhancement of durability is desired, due consideration shall be given to items such as reduction of water-cement ratio as well.

2) Allowable tensile unit stress of reinforcing bar

The allowable tensile unit stress of reinforcing bar is provided in Table 3.27. The allowable stress of reinforcing bar used for irrigation and drainage canals is determined, as a general rule, based on characteristics of the canal as described below.

Table 3.27 Allowable stress of reinforcing bar (Unit: N/mm2)

Type of reinforcing bar		Allowable tensile unit stress (σ_{sa}) For general members	For members determined by fatigue strength	For members in contact with water	For members subject to direct vehicle loads	Remarks
SR 235	During normal condition	137	137	137	137	
	During earthquake	205	205	205	205	
SD 295 A, B	During normal condition	176	157	157	137	
	During earthquake	264	264	264	264	
SD 345	During normal condition	196	176	176	137	
	During earthquake	294	294	294	294	

Note: (i) SD 345 can be used in structures when:
i) It is considered economically advantageous because the structure is particularly large, and conditions during earthquake are dominant factors,
ii) It is hard to obtain SD 295 A or B reinforcing bars due to market conditions,
iii) Adequate reviews for cracking, etc. would be performed, or
* The allowable tensile unit stress of SD 345 under normal condition shall be identical with that of SD 295
(ii) A or B when:
i) Reviews for deflection and cracking are not performed.

(a) Irrigation canal

The allowable tensile unit stress under the column “For members in contact with water” in Table 3.27 shall be applied.

(b) Drainage canal

The allowable tensile unit stress under the column “For general members” in Table 3.27 shall be applied. The allowable tensile unit stress under the column “For general members” shall be considered as a standard for reinforcing bars in irrigation canals or drainage canals when;

a) The cross section size of irrigation canal is small (the maximum dimension of width or height of the canal is approximately 1 m or less), or

b) They are appurtenant structures such as drops, confluence-diversion works, sluice/sluice pipe, etc. In addition, the allowable tensile unit stress under the column “For members in contact with water” in Table 3.27 shall be considered as a standard for reinforcing bars in drainage canals
when;

(i) The drainage canal is part of a dual-purpose canal,

(ii) It is considered appropriate to do so based on the site condition, importance, other characteristics of canal, or

(iii) It is determined to do so out of necessity as a result of consultations with other relevant authorities.

3) Unit stress of reinforcing bar for structures subject to inner water pressure

For structures subject to inner water pressure, it is recommended to reduce the allowable stress of reinforcing bars in order to secure water tightness by using Table 3.28 as reference.

[Reference]

In designing reinforced concrete member subject to tension, it is common to design in a way that allows reinforcing bars alone to bear the load without taking tensile stress of concrete into consideration. However, for structures such as siphon in which, in addition to its specific requirement for water tightness, tensile stress (hoop tension) is generated, causing commensurate stress in reinforcing bars, cracking occur in the concrete first because of the different creep ratios between the reinforcing bar and concrete. In order to limit those cracking in concrete within the allowable crack width defined by the structure type, durability, water tightness, covering thickness, etc., it is necessary to design in a way that the actual stress in reinforcing bar does not exceed a certain limit. In designing siphons, the allowable stress reduction formula has been established as a method to adjust the calculated stress values for reinforcing bar depending on the magnitude of inner water pressure applied and the shape of inner space.

The thickness of member shall be not less than 1/10 (one tenth) of the inner width for a box form siphon, and not less than the larger of 1/12 (one twelfth) of the inner width and 20 cm for a round form siphon. In addition, not only simply reducing the allowable stress of reinforcing bar, reduction of the stress applied to concrete by adjusting the concrete member thickness shall also be considered concurrently.

Table 3.28 Unit stress of reinforcing bar for structures subject to inner water pressure (Unit: N/mm^2)

Type	Allowable stress reduction formula	
	$\sigma_{sa} = 137^*$	$\sigma_{sa} = 157$
Box form	137-3.7H	157-4.0H
Round form	110-0.9H	127-1.0H

Note: Applicable ranges are H ≤10 m for box form type, and H ≤38 m for round form type. Herein, the notation H indicates the head (m) up to the center of the box or pipe which includes water hammer pressure.

* σ_{sa} = 137 N/mm^2 is applicable when vehicle loads are directly loaded.

4) Allowable stress in consideration of effects due to temperature change, drying shrinkage, earthquake, etc., as well as temporary loads

(a) When effects due to temperature change and drying shrinkage are considered, the allowable stress specified in "(1) Design strength and allowable stress of concrete" and "(2) Allowable tensile unit stress of reinforcing bar" can be increased by a factor of 1.15 at most.

(b) When effects due to earthquake are considered, the allowable stress specified in "(1) Design strength and allowable stress of concrete" and "(2) Allowable tensile unit stress of reinforcing bar" can be increased by a factor of 1.5 at most.

(c) When effects due to temperature change, drying shrinkage, and earthquake are considered, the allowable stress specified in "(1) Design strength and allowable stress of concrete" and "(2) Allowable tensile unit stress of reinforcing bar" can be increased by a factor of 1.65 at most.

(d) When effects due to temporary loads or extremely infrequent loads are considered, the allowable stress specified in "(1) Design strength and allowable stress of concrete" and "(2) Allowable tensile unit stress of reinforcing bar" can be increased. This provided that it does not exceed the value two times of "(1) Design strength and allowable stress of concrete," and the value 1.65 times of "(2) Allowable tensile unit stress of reinforcing bar."

(3) Design of reinforced concrete

Reinforced concrete structures shall be analyzed by the elastic theory, as a general rule, it shall be verified that the unit stress of reinforcing bar and concrete are not greater than the respective allowable stress.

1) Statically indeterminate force or elastic deformation

The radius of gyration of area and Young's modulus in the calculation of statically indeterminate force and elastic deformation are considered as follows.

(a) The calculation of radius of gyration of area shall be performed for the entire concrete cross section of the member with the effects of reinforcing bars taken into account. However, the calculation of statically indeterminate force can be performed for the entire concrete cross section of the member generally without considering the effects of reinforcing bars.

(b) The Young's modulus of reinforcing bar is Es = 200×10^3 kN/mm^2

(c) Values provided in Table 3.29 shall be considered as standards for the Young's modulus of concrete Ec.

Table 3.29 Young's modulus of concrete used in calculations of statically indeterminate force or elastic deformation

Design strength σ_{ck} (N/mm^2)	18	21	24	30	40
Young's modulus of concrete E_c (kN/mm^2)	22	23.5	25	28	31
Ratio of Young's modulus for reinforcing bar and concrete $n = E_s/E_c$	9.1	8.5	8.0	7.1	6.5

2) Cross section determination or unit stress calculation

In cross section determination or unit stress calculation, the tensile stress of concrete is generally disregarded, and the longitudinal strain is considered to be proportional to the distance from the neutral axis of cross section.

(a) The Young's modulus of reinforcing bar is Es = 200 × 10^3 kN/mm^2

(b) The ratio of Young's modulus for reinforcing bar and concrete in the allowable stress design method shall be provided by n = E s / Ec = 15, as in the established practice.

(c) When the reinforcing bar is not intersecting perpendicular to the design cross section, multiplying the cross sectional area of reinforcing bar by sine of the angle at which the reinforcing bar intersects with the cross section gives the value for effective cross section of reinforcing bar.

3) Calculation of cross section and unit stress

(a) Rectangular cross section subject to bending moment only

Calculations of unit stress and resisting moment shall be performed by using the equations (F. 3.55) to (F. 3.63) for the rectangular cross section member subject to bending moment only.

a) Explanation of symbols used in calculations (Units of quantity used are N and mm, or kN and m.

h: Full height of cross section

b: Width of cross section

d_1: Covering depth

σ_{ca}: Allowable compressive unit stress due to bending of concrete

σ_{sa}: Allowable tensile unit stress of reinforcing bar

n: The ratio of Young's modulus for reinforcing bar and concrete E s / Ec = 15

d_0: Effective depth of balanced cross section

d: Effective depth (= h − d1)

M: Bending moment due to external force

S: Shearing force

A_s: Cross sectional area of reinforcing bar

τ : Shear unit stress of concrete

τ_o : Bond stress of concrete

U: Total sum of bar perimeters

M_{rc}: Resisting moment of concrete

M_{rs}: Resisting moment of reinforcing bar

k_0 and j_0 used in cross section calculation.

$$k_0 = \frac{1}{1+\frac{\sigma_{sa}}{n\sigma_{ca}}} \qquad j_0 = 1 - \frac{k_0}{3}$$

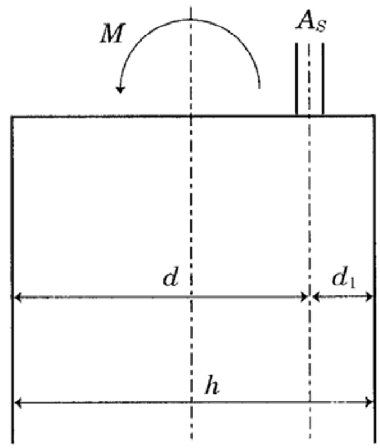


Fig. 3.22

Illustrative diagram of cross section

k and j used in unit stress calculation.

$$k = \sqrt{2n \cdot p + (n \cdot p)^2} - n \cdot p \qquad j = 1 - \frac{k_0}{3}$$

where p: Reinforcement ratio $\qquad p = \frac{A_s}{b \cdot d}$

b) Calculation formula

$$d_0 = \sqrt{\frac{M}{\frac{1}{2} \cdot b \cdot \sigma_{ca} \cdot k_0 \cdot j_0}} = C_1\sqrt{\frac{M}{b}} \quad \text{(F. 3.55 a)}$$

$$C_1 = \sqrt{\frac{2}{\sigma_{ca} \cdot k_0 \cdot j_0}} \quad \text{(F. 3.55 b)}$$

$$A_s = C_2\sqrt{M \cdot b} \quad \text{(F. 3.56 a)}$$

$$C_2 = \frac{\sigma_{ca}}{2\,\sigma_{sa}} \times \sqrt{\frac{6\,n}{(2\,n \cdot \sigma_{ca} + 3\,\sigma_{sa})}} \quad \text{(F. 3.56 b)}$$

$$A_s = \frac{M}{\sigma_{sa} \cdot j_0 \cdot d} \quad \text{(F. 3.57)}$$

$$\sigma_c = \frac{2\,M}{k \cdot j \cdot b \cdot d^2} \quad \text{(F. 3.58)}$$

$$\sigma_s = \frac{M}{A_s \cdot j \cdot d} = \frac{M}{p \cdot j \cdot b \cdot d^2} \quad \text{(F. 3.59)}$$

$$\tau = \frac{S}{b \cdot j \cdot d} \quad \text{(F. 3.60)}$$

$$\tau_0 = \frac{S}{U \cdot j \cdot d} \quad \text{(F. 3.61)}$$

$$M_{rc} = \frac{\sigma_{ca}}{2} \cdot k \cdot b \cdot j \cdot d^2 \quad \text{(F. 3.62)}$$

$$M_{rs} = \sigma_{sa} \cdot p \cdot b \cdot j \cdot d^2 \quad \text{(F. 3.63)}$$

4) Amount of distribution reinforcement and minimum or temperature bar

(a) Role of distribution reinforcement

Distribution reinforcement is the reinforcing bar, which is placed perpendicular to positive or negative reinforcement in order to distribute the bending moment and shearing force due to concentrated load or partial distributed load. The distribution reinforcement is placed perpendicular to the main reinforcement of slab-form structures (e.g. slab, canal, retaining wall, box culvert, etc.).

(b) Amount of distribution reinforcement

The amount of distribution reinforcement placed perpendicular to the main reinforcement in the standing wall or base of retaining wall and canal shall be one sixth of the main reinforcement or more but not less than 500 mm^2/m, in accordance with the standard for one-way slab system.

(c) Amount of minimum or temperature bar

The minimum area of main reinforcement or the area of temperature bar can be 500 mm^2/m.

5) Precaution reinforcement

(a) Role of precaution reinforcement

Precaution reinforcement is the reinforcement placed for precautions against stress concentration due to loads, and against cracking due to temperature and drying shrinkage. It is normally placed close to the exposed large surfaces and around openings of slab-form structures (e.g. slab, canal, retaining wall, box culvert, etc.), and in areas subject to concentrated load. It serves to reinforce the areas where the amount of reinforcement cannot be defined by normal structural calculations, in order to prevent anticipated harmful cracking from occurring. In retaining walls, it is the reinforcement placed horizontally in the front side surface exposed to direct sunlight.

(b) Amount of precaution reinforcement

Descriptions in Table 3.30 shall be used as a guideline for the amount of precaution reinforcement.

Table 3.30 Amount of precaution reinforcement

Structure	Amount of precaution reinforcement
Concrete surface with large exposed area, such as retaining wall, etc.	Place the reinforcement of 500 mm^2 or more per meter in horizontal direction with the center-to-center spacing of 300 mm or less, close to the exposed side surface of the wall.
Box culvert, etc. Structures with two sides constrained	Place the reinforcement of approximately 500 mm^2 per meter in horizontal direction close to the both side surfaces of the wall.

6) Reinforcement clearance

While reinforcement clearance varies depending on the type and size of member, the maximum dimension of aggregate, the size of reinforcing bar, etc., it shall be defined in consideration of reinforcement placement, concrete placement, bond strength between reinforcement and concrete, etc.

(a) The horizontal clearance between the reinforcing bars in beam members shall be not less than 20 mm, 4/3 times or more the maximum dimension of coarse aggregate, and equal to or greater than the diameter of the reinforcing bar. In addition, when positive or negative reinforcement is placed in multiple layers, the vertical clearance shall be generally not less than 20 mm and equal to or greater than the diameter of the reinforcing bar.

(b) The center-to-center spacing between the slab main reinforcing bars shall be two times or less the slab thickness and not more than 300 mm when maximum bending moment is applied on the cross section of the slab. It is desirable that the spacing is three times or less the slab thickness and not more than 400 mm on others cross-sections.

(c) The clearance for axial reinforcement in a column shall be not less than the largest of 40 mm, 4/3 times of the maximum dimension of coarse aggregate, and 1.5 times of the diameter of reinforcing bar.

(d) To define the reinforcement clearance when deformed reinforcing bars are placed in bundle, the criteria 1 ~ 3 described above shall be applied to the hypothetical reinforcing bar which cross sectional area is equivalent to the sum of the cross sectional areas of bundled bars.

7) Reinforcing bar covering

While the reinforcing bar covering shall be defined in accordance with the importance of structure, site conditions, etc.

The "covering" means the shortest distance between the reinforcing bar surface to the concrete surface, unless otherwise specifically prescribed.

(a) While the covering shall be not less than the diameter of reinforcing bar, it is generally not less than the value provided in Table 3.31.

Table 3.31 Minimum covering of reinforced concrete (Unit: mm^2)

Environment condition \ Member	Slab	Beam	Column
General environment	25	30	35
Corrosive environment	40	50	60
Particularly severe corrosive environment	50	60	70

(b) Exterior covering of the base slab of structure such as footing, flume, culvert, siphon, etc., which are directly placed under ground, is required to be not less than 75 mm.
(c) In the cases other than particularly severe corrosive environment, values provided in Table 3.32 shall be considered as a standard for the distance from the center of main reinforcement to the concrete surface except for the case of column,
However, when the size of main reinforcement is not larger than 13 mm in a small-scale structure, the distance from the center of main reinforcement to the concrete surface can be 50 mm.

Table 3.32 Standard values of reinforcement covering (Unit: mm)

Construction condition \ Diameter of main reinforcement	Not more than 19 mm	Not less than 22 mm	Remarks
Concrete surface finished by formwork or screening	60	70	Member thickness T < 300 mm
	70	70	Member thickness T ≥ 300 mm
Bottom surface of base, which is directly placed under ground.	90	100	Footing, flume, culvert, siphon, etc.
Over pile top	50	50	In the case of the pile top penetrating into the base 50 mm or more

(d) The reinforcement covering of reinforced concrete placed underwater shall be not less than 100 mm.
(e) The reinforcement covering for parts where there is a potential for wear due to water flow and others shall be increased accordingly.
(f) To define the reinforcement covering when deformed reinforcing bars are placed in bundle, the criteria 1 to 5 described above shall be applied by assuming the bundled bars as a single reinforcing bar.

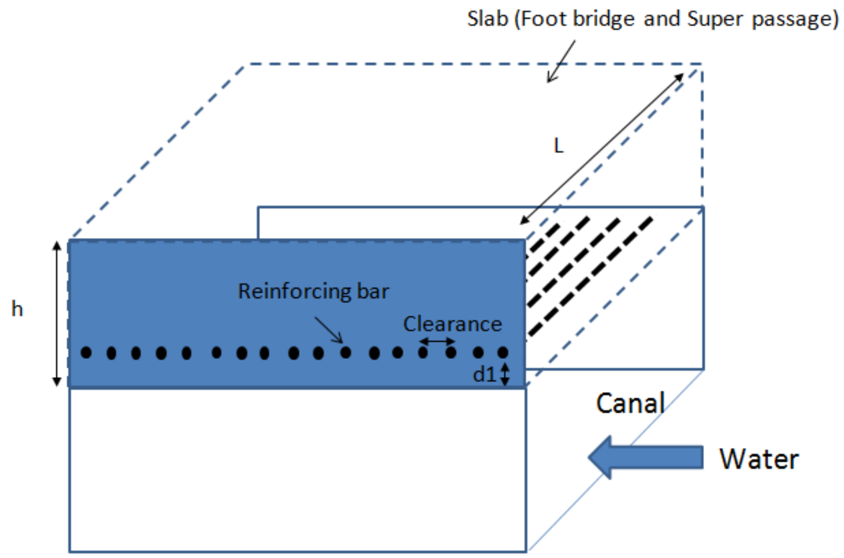
8) Recommendable dimension of reinforced concrete

The recommendable dimension of reinforced concrete which is constructed as slab for Foot bridge or Super passage is as follows,
(a) In case of the load on slab which is considered human and livestock

L (m)	h (m)	d1 (m)	Clearance (m)	Size of reinforcing bar
～1.5	0.13	0.06	0.25	D12
1.6～2.5	0.2	0.06	0.25	D12
2.6～3.5	0.2	0.06	0.15	D12
3.6～4.5	0.2	0.06	0.15	D16

(b) In case of the load on slab which is considered a car (10t truck)

L (m)	h (m)	d1 (m)	Clearance (m)	Size of reinforcing bar
～1.5	0.25	0.06	0.15	D12
1.6～2.5	0.25	0.06	0.15	D16
2.6～3.5	0.25	0.06	0.10	D16
3.6～4.5	0.25	0.06	0.10	D20



(4) Minimum thickness of members

The minimum thickness of members used in canal structures shall generally be determined in consideration of workability and water tightness requirements. For good workability, vertical walls of 2.0m or taller that use double reinforcement shall have a minimum thickness of 20cm, and vertical walls shorter than 2.0m using single reinforcement shall have a minimum thickness of 13cm. Siphons and other structures subject to water pressure shall have a minimum thickness of 20cm to ensure good workability and water tightness.

(5) Joints

Joints shall be provided at suitable positions in suitable intervals in accordance with the type and size of the structure, the site and the building conditions.

1) Construction joints

Construction joints will be required where the type of the structure, size, or other conditions for work hinder continuous placement of concrete. Position and structure of construction joints shall be indicated in the design drawing in principle.

(a) Positions of construction joints shall be determined according to the type and size of the structure and other conditions for work. Structurally, construction joints should be provided at positions subject to the lowest possible interval stress.

(b) Grooves shall be formed if higher safety against shear force is desired. Structures subject to internal water pressure (e.g. siphons) water-stops shall be fitted to the construction joints perpendicular to the direction of water flow.

2) Contraction joints

Contraction joints shall be provided to prevent cracking of the concrete due to contraction. Positions and construction of contraction joints shall be indicated clearly on the design drawing. Generally, contraction joints are used in siphons, culverts, tunnels and buried structures. Positions of construction joints shall be determined according to the type of structure, foundations, and other conditions for work. Contraction joints should be provided at standard intervals of 3 to 5m in thin concrete linings of 10cm or less, and in other canal structures at intervals of 9m. Pedestals or dowel bars shall be provided to prevent movement of the structure due to uneven settlement or earthquake.

3) Expansion joints

Expansion joints shall be provided to prevent cracking of the concrete due to expansion. Expansion joints are usually used in structures exposed above ground.

(a) The position of expansion joints shall be determined according to the type of structure, foundations and other conditions for work. Expansion joints shall be provided at points where the construction, section, or form changes in principle (for example, before and after an open transition where the construction changes). For a series of structures exposed above ground such as flumes, expansion joints shall be provided at intervals of 24m.

(b) Expansion plates shall be used instead into void of the joints. Pedestals or dowel bars shall be provided to prevent movement of the structure due to uneven settlement or earthquake.

(6) Cut-off walls

Cut-off walls shall be provided generally at both ends of an open transition connected to an open channel to prevent seepage of water along the periphery of the structure.

1) Structure and standard dimensions are shown in Table 3.33.

2) Vertical reinforcing bars of the same size as the bar used in the open transition shall be inserted into the center of the wall.

Table 3.33 Standard dimensions for cut-off wall (Unit: mm)

Water depth	Depth × Width	Thickness
900 or less	600 × 600	150
More than 900 but less than 1,800	750 × 750	200
1,800 or more	900 × 900	200
Small structures	450 × 450	150

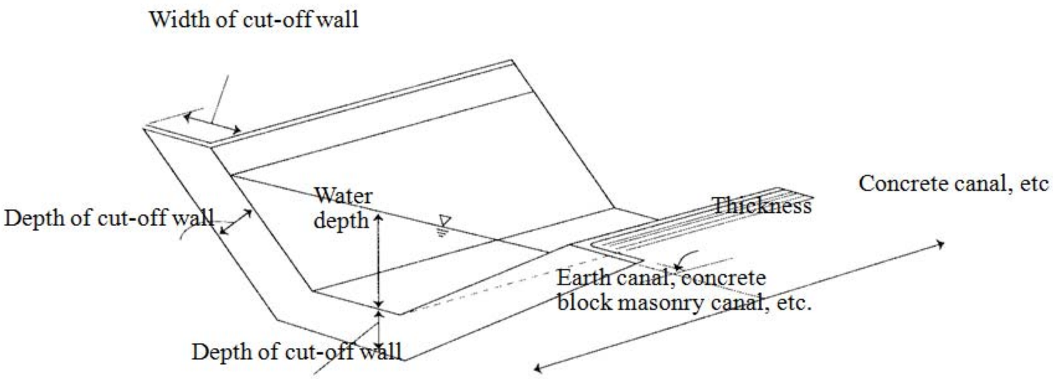


Fig. 3.33 Structure of cut-off wall

(7) Haunches

Haunches shall be provided on square or acute-angled corners of the structure to prevent concentration of stress there and to increase the strength of the structural members. The values of Table 3.34 shall be used as the standard according to the size of the box section and the height of the vertical retaining wall.

Table 3.34 Size of haunch (Unit: mm)

Dimension of box cross section	Height of vertical retaining wall	Size of haunch
Less than 1,000	Less than 1,000	–
1,000 or more but less than 2,500	1,000 or more but less than 2,500	150 × 150
2,500 or more	2,500 or more	200 × 200

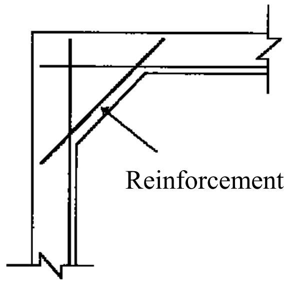


Fig. 3.34 Haunch in rigid-frame and inner side reinforcement along haunch

3.4 Detail designs for canals and related structures

3.4.1 Basics of Design

(1) Cross-sectional forms of open channels (Hydraulically favorable cross-sections)

Before determining cross-sectional forms of open channels, favorable cross-sectional forms from hydraulic, structural, and constructional aspects shall be studied, so that the most cost effective cross-sectional forms should be selected.

When a flow area A is given, a channel cross-section with the shortest length of wetted perimeter p is capable for the largest quantity of flow. Such a cross-section is called the most hydraulically effective cross-section, and it is generally accepted that cross-sections close to the most effective cross-section are usually most cost effective as well. The most hydraulically effective cross-sections for trapezoid and rectangular cross-sections are obtained through Equation (F. 3.64) and Equation (F. 3.65).

$$b = 2H \times \tan(\theta/2) \quad \text{……………………..(F. 3.64)}$$

$$H = \sqrt{A\sin\theta / (2-\cos\theta)} \quad \text{……………………..(F. 3.65)}$$

Where
- H: Water depth (m)
- B: Width of water surface (m)
- b: Width of channel bed (m)
- θ: Angle between sidewall and horizontal plane (°)
- m: Gradient of slope (cotθ)
- ℓ: Slope length (m)
- A: Cross-sectional flow area (m^2)

The most hydraulically effective cross-sections for each side slope gradient m in channels of trapezoid and rectangular cross-section are as shown in Table 3.35.

Table 3.35 The most hydraulically effective cross-sections and dimensions

m	0.0	0.3	0.5	0.57	1.0	1.25	1.5	2.0
θ	90°	73°18'	63°26'	53°08'	45°	38°40'	33°41'	26°34'
$\sin\theta$	1.000	0.958	0.894	0.800	0.707	0.625	0.555	0.447
$\cos\theta$	0.000	0.287	0.447	0.600	0.707	0.781	0.832	0.894
ℓ/H	1.000	1.044	1.118	1.250	1.414	1.600	1.803	2.236
B/H	2.000	2.088	2.236	2.500	2.828	3.200	3.606	4.472
b/H	2.000	1.488	1.236	1.000	0.828	0.702	0.606	0.472
$H/\sqrt{A}$	0.707	0.748	0.759	0.756	0.739	0.716	0.689	0.636

(2) Freeboard

The freeboard of canal shall be determined in consideration of the canal elements such as purposes, cross section shape, route shape, size, importance, site conditions, allocation of canal structures, flow velocities, etc.

1) Factors for determining canal freeboard

(a) Coefficient of roughness

The coefficient of roughness of canal surface varies widely depending on construction and arrangement of canal even in the light of actual measurement results. Although it would be safe if the whole amount which covers potential fluctuation is given as a factor of freeboard, there are actual instances where fluctuation of approximately "n = 0.001" is incorporated for concrete lining, etc., based on probability of occurrence and uncertainties associated with fluctuation range, etc. While necessary freeboard based on coefficient of roughness also varies depending on materials, cross section shape, and other elements of canal, approximately 5 to 7 % of the water depth is commonly used for concrete canals.

(b) Velocity head

Since the flow running down the canal is in moving condition, it always has velocity head. It has potential to raise the water surface as it changes to static head. When there is no structure or other factors that hinder the water flow inside the canal for a typical open channel, it is very unlikely to see the uprise of water surface due to velocity head. However, when there are gates and/or screens that can block the canal completely, it is possible to have an uprise of water surface by 50 to 100 % of the velocity head in the upstream side of these structures depending on their operation mode. Like this, since the height of water surface uprise to be considered as the freeboard varies widely depending on the degree of hindrances in the canal, 50 % of the velocity head shall be allotted to deal with unanticipated situations. The other events for canals where factors for water surface uprise due to velocity head is not particularly expected, and 100 % shall be allotted when water surface uprise is expected due to the size and importance of canal, allocation of structures, etc. Additionally, when there are facilities such as spillways, bypasses, etc. that restrains the water level from rising, 50 % of the velocity head shall be anticipated.

(c) Water surface vibration

The flow in a canal generates wave movement due to structures (gates, drops, chutes, pumping stations, etc.), wind, etc., resulting in water surface vibration. While the degree of water surface vibration varies with 1 the arrangement of structures in the canal, 2 relationship between wind direction and the canal orientation, 3 the water surface width, and 4 the water depth, it is normally considered to be approximately 10 to 30cm. Therefore, the half-wave height of such vibration, that can be 5 to 15 cm depending on the canal conditions, shall be added as freeboard specifically for the water surface vibration.

(d) Freeboard by discharge ratio

The freeboard is to deal with unanticipated events that a canal may encounter, and its size shall be assessed by the discharge, which is capable of flow including such freeboard. The ratio of the possible discharge in cross section including freeboard and the design discharge (the possible discharge in cross section including freeboard divided by the design discharge) shall be approximately 1.25 to 1.35, and that ratio shall not be below 1.2.

2) Points to be looked into to determine canal freeboard

While canal freeboard shall be assessed based on each factor described above, it is necessary to study each case in design to incorporate modifications in accordance with the field conditions. The following are points to be looked into when doing so.

(a) Size, importance, and site conditions

The size and importance of canal must be considered in determining the freeboard. It is inappropriate to treat an important main canal involving a wide area and other main/lateral canals, diversion canals or secondary canals equally. Similarly, a canal on embankment close to housing area may have freeboard somewhat different from that of an intermountain canal.

(b) Types of canal structure

The adaptability of a canal for unanticipated events varies with types of canal structure and cross section shape of water conveying facility. In internal pressure siphons, tunnels, circular or horseshoe shape culverts, the relations applied to the head increase or water conveying capacity increase changes once a certain limit is exceeded. Therefore, types of structure, allocation, hydraulic characteristics of the canal should be also considered in determining the freeboard, and it is necessary to carefully determine the freeboard together with consideration of spillways, etc., for the canal portion directly upstream of these canal structures.

(c) Allocation of structures and canal curvature

Structures (drops, chutes, gates, screens, etc.) within a canal and sharp curvature of a canal cause backwater or wave movement. Thus, in allotting freeboard, relationships with these subjects shall be also considered, and there can be instances that freeboard beyond its standard value for some canals.

(d) Operation

Depending on possible change in water source discharge level, structures and operation of diversion works and spillways, a flow beyond the design discharge level may run down the canal. Like this situation, particularly in case of canal, these factors must be taken into account in allotting freeboard for areas close to the intake.

(e) Flood inflow

Under the concept of this technical guideline, flood is not expected to flow into the canal as a general rule for an irrigation canal. However, for situations where flood flow of a certain basin is brought in the canal out of necessity, or for the storm water which drops and flow into the canal lot, their water volumes have to be considered in determining freeboard. In this case, it is desirable to leave freeboard of approximately 10 cm from the lining top. Also, there have been reported cases noting that coefficient of roughness is significantly increased due to overgrowth of water plants on the canal inside surfaces. While these water plants can be removed by routine maintenance and operation efforts, the task is sometimes too difficult. Thus, when it is assessed imperative, estimate appropriate coefficient of roughness to take measures such as increasing freeboard, etc.

3) Calculation methods for canal freeboard

Standard calculation methods for canal freeboard by purposes of canal, by types of canal, and by cross section shapes are as follows.

(a) Basic equation for canal freeboard calculation ($Fr \leq 1$)

The basic equations for canal freeboard calculation by cross section shapes and types of canal are as follows.

a) Non-lining canals and lining canals

The freeboard for non-lining canals and lining canals shall be calculated by using the equation (F. 3.66), as a general rule.

$$F_b = 0.05\, d + \beta \cdot h_v + h_w \quad \text{(F. 3.66)}$$

Where F_b: Freeboard (m)
d: Water depth corresponding to design discharge (m)
h_v: Velocity head (m)
β: Conversion factor from velocity head to static head, ranging 0.5 - 1.0
h_w: Freeboard for water surface vibration (m)

b) Retaining wall canals (flumes, retaining wall canals, box culverts, ready-made product canals, etc.) The freeboard for retaining wall canals shall be calculated by using the equation (F. 3.67), as a general rule.

$$F_b = 0.07\, d + \beta \cdot h_v + h_w \quad \text{(F. 3.67)}$$

(b) Calculations of freeboard by canal purposes and canal sidewall height

Calculations of freeboard and sidewall height for the open channel irrigation or drainage canals shall be performed according to the flow charts shown below.

The freeboard for irrigation or drainage canals is the height indicated in Fig. 3.35.

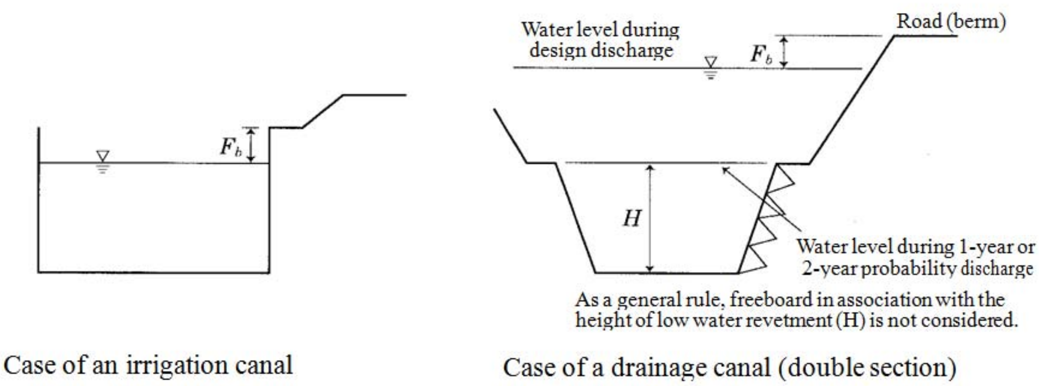


Fig. 3.35 Freeboard for canals

a) Irrigation canals

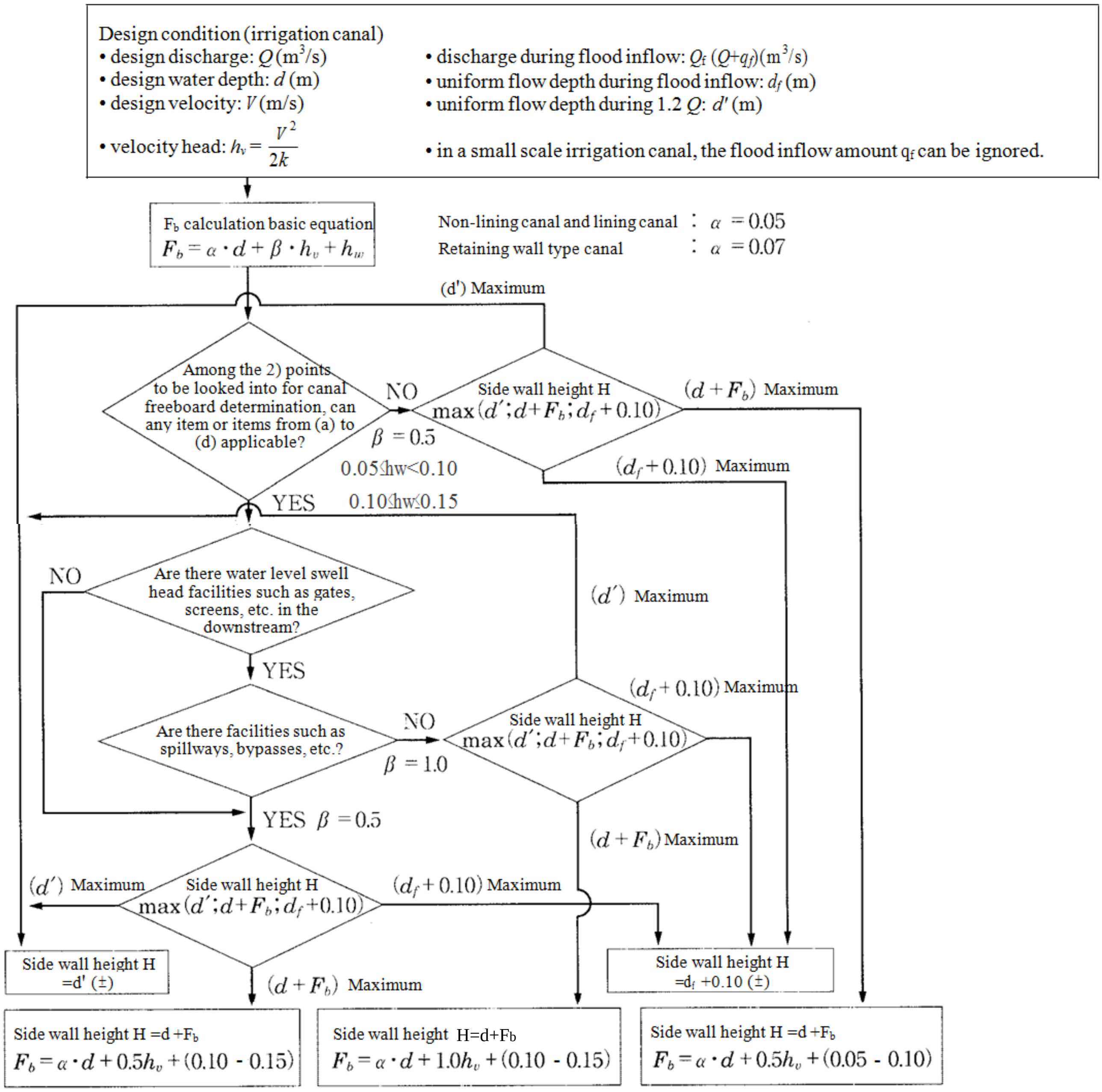
Calculations of freeboard and sidewall height of irrigation canals including non-lining canals, lining canals (trapezoidal cross section canals), and retaining wall canals (flumes, retaining wall canals, box culverts, ready-made product canals, etc.) shall be performed in accordance with the flow chart shown in Fig. 3.36.

b) Drainage canals

Calculations of freeboard and sidewall height of drainage canals including non-lining canals, lining canals (trapezoidal cross section canals), and retaining wall canals (flumes, retaining wall canals, box culverts, ready-made product canals, etc.) shall be performed in accordance with the flow chart shown in Fig. 3.37.

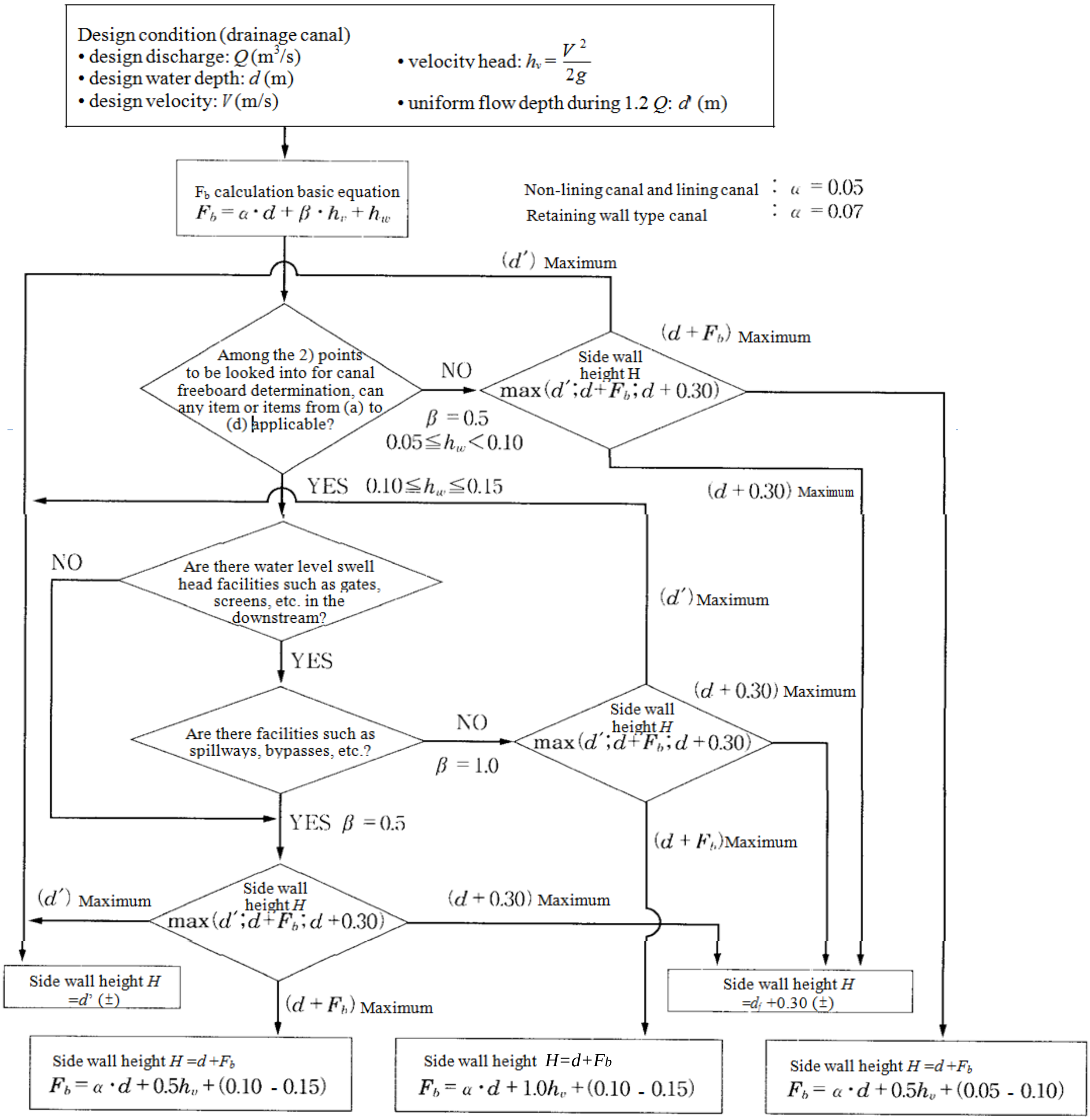
c) Dual-purpose canals

The sidewall height of dual-purpose canals shall be calculated by using the freeboard obtained for either 1 Irrigation canals or 2 Drainage canals, whichever is greater. Also, for a typical open channel type canal in which allocation of structures or hydraulic conditions that cause significant water level uprise due to the velocity head are not expected, it is considered sufficient to allot 0.5 hv which can be derived from assumptions already described or judgment based on experience. Furthermore, particularly in cases that water level uprise are expected and bypasses or spillways are installed as countermeasures against them, again, 0.5 hv shall be used.



Note: In cases of (b) or (c) of the points to be looked into for canal freeboard determination, if it is deemed necessary by a hydraulic study, the sidewall height may be calculated by means other than described above.

Fig. 3.36 Flow chart for calculation of irrigation (open channel) canal freeboard and determination of canal sidewall height



Notes: 1. In cases of (b) or (c) of the points to be looked into for canal freeboard determination, if it is deemed necessary based on a hydraulic study, the side wall height may be calculated by means other than described above.

2. For small-scale drainage canals, the minimum freeboard 0.30 m can be reduced.

Fig. 3.37 Flow chart for calculation of drainage (open channel) canal freeboard and determination of canal sidewall height

(c) Freeboard of supercritical or rapid flow canals (Fr > 1)

The freeboard of a steep slope canal with rectangular cross section where the running water becomes the supercritical flow shall be normally calculated by using equation (F. 3.68), as a general rule.

$$F_b = 0.6 + 0.037\, v \bullet h^{1/3} \qquad \text{(F. 3.68)}$$

Where F_b: Freeboard (m)

v: Velocity (m/s)

h: Water depth (m)

Herein, that Fb and h are distance measured perpendicular to the steep slope canal bed.

Since the design of freeboard for a steep slope canal varies widely depending on factors including the size, gradient, cross section shape, degree of unevenness of canal, in addition to the equation (F. 3.68), It is necessary to study the water surface uprise due to air entrainment by the high velocity supercritical flow. The effects of the roll wave trains on the water surface vibration caused by waves, and the splash height generated by the unevenness of canal.

Additionally, for the case of a small scale supercritical or rapid flow canal, since the freeboard calculated by using the equation (F. 3.68) would be an over estimate, it is necessary to study the freeboard otherwise by using the equation (F. 3.69) or (F. 3.70) which is the freeboard calculation formula, or other means.

$$F_b = CVh^{1/2} \qquad \text{(F. 3.69)}$$

$$F_b = 0.6 + 0.037\, Vh^{1/3} \qquad \text{(F. 3.70)}$$

Where

F_b: Freeboard (m)

V: Velocity (m/s)

h: Water depth (m)

C: Coefficient Rectangular canal 0.1, trapezoidal canal 0.13

Herein, both the water depth and the freeboard shall be perpendicular to the inclination of the bottom of the steep slope distribution canal.

(3) Treatment for the foundation ground

Based on the foundation ground conditions that have been grasped through soil investigations, safety of the channel structures should be studied so that an appropriate design should be performed. When unfavorable ground exists in the foundation of the channel, it is normally preferable to make a plan avoiding such a site. However, if it is inevitable to make such a plan due to various conditions including cost efficiency of the alignment of the channel or ease of works, then suitable soil stabilization works or foundation works should be selected. So that the design should be executed to take counter measures for preventing differential settlement and others, or, for important channels, even liquefaction on the occasion of earthquakes, etc.

In channel construction, however, changes in the foundation ground are often realized in the site working stage, therefore supplement investigations or the like should be executed for such occurrences, and it is required to cope with quickly basing on the investigation results.

1) Soft ground

As for conditions that lead to ground settlement or sliding due to soft ground, the following can be noted: increase of contact pressure by surcharge load, a consolidation phenomenon in the ground, as well as liquefaction in the loose sandy ground, pumping up groundwater, or even degradation activity on organic matter, etc. Thus, it should be decided how to deal with the issue, after discussing a long-term safety for settlement and sliding. Especially for designs near the boundary of the soft ground and otherwise, careful attention should be paid for not only foundation treatment but also the structure itself of the open channel.

2) Expansive clay

In cases where consolidated clay exists in the foundation of the linings, the clay generally absorbs water and expands (swells) after starting water flow operation, sometimes causing excess pressure onto the linings to damage through heaving. In addition, the swelling may result in significant reduction in shear strength of the clay that can make the slopes slide. Although it is not so often to encounter such excessively dried and consolidated clay, careful attention should be paid to clay in deep layers in cases of large cutting. There are two methods for treating expansive clay: to spray water for allowing the clay to expand before lining works, and to replace with good quality materials. In either case, the property of the expansive clay should in advance be thoroughly grasped, and discussions should be made on the safety of the structures.

3) Boulders and Rock Mass

In cases where the foundation ground is formed by boulder stones or partly by rock mass, loads on channel structures tend to concentrate to this part, often causing to develop cracks or the like in the structures. Hence, it is significant to take proper measures such as removing boulders, cutting down further, or replacing the ground, etc. In cases of continuous rock mass especially near the boundary of compressible ground, the design is required careful attention to structures, joints, etc. of the open channel as well as to their behaviors during earthquakes. Additionally, it is necessary to be aware that the concepts for ground reaction or structural analysis methods are different in rock foundations and usual soil foundations.

(4) Earth works

In planning earth works for open channels, an appropriate scheme of haul excavation and filling shall be established through studying balance of cutting/filling earth volumes, reusing, waste, etc., and types of structure, construction scale, working conditions, or the like shall be considered.

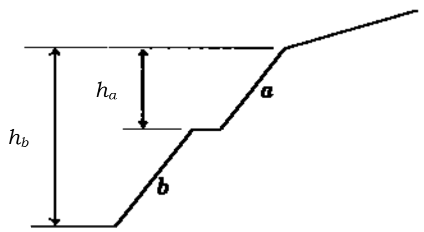
1) Cutting

Natural ground often shows non-uniformity with reasons including that soil property significantly differs depending on the extent of weathering and cracking, cutting height, state of spring water, conditions of stratification, water content, etc. Therefore, for designing gradient of slopes, investigations should be in advance carried out for site conditions, state of existing slopes, or the like, and then design specifications, for including cutting heights or slope gradient suitable to each site condition, should be determined referring to Table 3.36 or actual values in the past, etc. In this connection, stability calculations shall be performed when specifically required. When the cutting height is large, berms should be provided depending on the state of soil character and the site conditions. As the sizes of berms, actual examples in the past show that, many of them have berm width of 0.5 to 1.0 m for cutting height 5 m. In cases where rainwater can possibly scour or slope failure, it is required to provide collecting/draining trenches along the top of slopes and berms, so that surface water should escape.

Table 3.36 Standard gradient of cutting slope Note 1)

Soil property of natural ground		Cutting height	Gradient
Hard rock			1 : 0.3 - 1 : 0.8
Soft rock			1 : 0.5 - 1 : 1.2
Sand	Not dense and gradation distribution is bad		1 : 1.5 -
Sandy soil	Dense (tight)	5 m or less	1 : 0.8 - 1 : 1.0
		5 - 10 m	1 : 1.0 - 1 : 1.2
	Not dense (loose)	5 m or less	1 : 1.0 - 1 : 1.2
		5 - 10 m	1 : 1.2 - 1 : 1.5
Pebbly soil, gravel or rock-lump mixed sandy soil	Dense or gradation distribution is good	10 m or less	1 : 0.8 - 1 : 1.0
		10 - 15 m	1 : 1.0 - 1 : 1.2
	Not dense or gradation distribution is bad	10 m or less	1 : 1.0 - 1 : 1.2
		10 - 15 m	1 : 1.2 - 1 : 1.5
Cohesive soil Note 2)		10 m or less	1 : 0.8 - 1 : 1.2
Rock-lump or cobble stone mixed cohesive soil		5 m or less	1 : 1.0 - 1 : 1.2
		5 - 10 m	1 : 1.2 - 1 : 1.5

Notes: 1. For the concept of the cutting height and gradient which is not a single gradient due to soil configuration, refer to Fig. 3.38.
2. Silt is included in cohesive soil.
3. Soil properties other than the above will be considered separately.



1. Gradient does not include berm.
2. The cutting height to the gradient shall be the entire upper cutting height from the related cutting slope.
h_a: Cutting height to slope a
h_b: Cutting height to slope b

Fig. 3.38 Cutting height and gradient

2) Banking

For designing banking, studies are required as shown below.

(a) The type of banking shall be decided depending on structures of the open channel, topographic features, presence or absence of soil to be reused, balance of cut and banking, etc. Especially for lining canals where stability of structures heavily depends on the banking conditions, it is required not only to fill up good quality soil and compact it sufficiently so that rainwater should not enter into the backside of linings, but also to carry out a careful design. It includes securing necessary banking height and crest length with considerations for maintenance at the top of the linings, etc.

(b) Good or poor quality of banking materials has great effect on ease or difficulty of works, as well as on stability of the banking after completion. In open channel construction, it is necessary to establish the most economical earthwork plan as possible through studies including properly reuse the materials, which have been developed through cutting, for banking materials.

When it is difficult to obtain appropriate materials, note that it can be another option to rethink the type of the structure. Suitability of banking materials is judged depending on using positions, banking heights, banking types, and working methods, etc.

(c) Gradients of banking slopes cannot be defined sweepingly as they depend on the site topography, banking materials, banking methods, slope protection methods, etc. Design specifications, for including banking heights or gradients of banking slopes suitable to each site condition, should be determined referring to Table 3.37 or actual values in the past. When specifically required, stability calculations shall be performed to decide the design specifications.

Table 3.37 Standard gradient of slope related to banking material and banking height

Banking material	Banking height	Gradient	Notes
Sand with good gradation (S), pebble, and fine-grain mixed pebble (G)	5 m or less	1 : 1.5 - 1 : 1.8	Applied to the banking whose base ground provides sufficient bearing capacity that will not be affected by water exposure. Unified classification in parentheses indicates a representative example.
	5 - 10 m	1 : 1.8 - 1 : 2.0	
Sand with bad gradation (SG)	10 m or less	1 : 1.8 - 1 : 2.0	
Rock lump (including muck)	10 m or less	1 : 1.5 - 1 : 1.8	
	10 - 20 m	1 : 1.8 - 1 : 2.0	
Sandy soil (SF), hard viscous soil, hard clay (diluvium hard viscous soil, clay, *Kanto* loam, etc.)	5 m or less	1 : 1.5 - 1 : 1.8	
	5 - 10 m	1 : 1.8 - 1 : 2.0	
Volcanic cohesive soil (V)	5 m or less	1 : 1.8 - 1 : 2.0	

Note: The banking height is a height difference between the top of slope and the toe of slope.

3) Crest length

Crest lengths of both sides along cross-sectional area of flow shall be decided through considering to secure the channel safety as a matter of course, and also considering necessary width for maintenance and repair works of the channel, cost effectiveness, etc. In some cases, the top is thus designed to provide with functions as a maintenance road enabling to inspect the channel or to operate gates, etc. When fences or guard rails are set at the top or near the top of the channel, structural consideration is required for the channel, and when a side drain or the like is placed, the design should take the placing width for it into account.

3.4.2 Masonry Canal

(1) Outline for applications

A masonry canal is a type of canals where the wall body is independent from the canal base, and the retaining wall itself acts a role for maintaining stability against earth pressure from behind, water pressure, inner water pressure, etc. For this canal, a large bearing capacity is necessary comparing with flumes or others, and therefore in cases where the foundation ground is soft, discussions are required on replacement with good quality materials or foundation treatment including pile foundations before making a decision for adopting or not adopting.

(2) Types of canal

The type of masonry canals is gravity type. Masonry canals are generally of stone and mortar structures that act as earth retaining structures as well, and are often used for drainage and irrigation canals, where wall heights are less than 2 m.

(3) Structural designs

In structural designs for masonry canals, loads to be given shall be determined conforming to " 3.3.4 Loads acting on open channels" and member dimensions and stress intensity of each part of the canal structures shall be studied, and also stability of the canal shall be discussed. Specific descriptions of the studies are as follows.

1) Studies on overturning, sliding, and bearing capacity of the foundation ground

The details are shown in "3.3.5 Stability analysis"

2) Joints

Joints are provided in response to structural and constructional necessity, similarly as in the case of flumes.

3) Weep holes and underdrains

Weep holes and underdrains are placed for the purpose of lowering outside water pressure that acts on canal sidewalls, as well as reducing the risk of floating up of canals.

3.4.3 Concrete Lining Canals

(1) Outline for applications

A concrete lining canal is a canal where concrete is used for a pavement material. Concrete has higher resistance against corrosion than other generally used materials, and is adaptable to comparatively large velocity. Concrete linings, especially those designed and constructed favorably, have small roughness, and are broadly applied. Since stability of canals of this type depends on the stability of the foundation ground, the lining thickness generally requires only the bare minimum. However, in cases including where the groundwater level is high and amount of spring water is large, or the foundation ground is unsuitable, the canal is structurally unstable and consequently it is desirable to discuss on structures or to select other construction method.

(2) Types of concrete lining canal

As representative types of concrete lining canal, there are concrete lining canals and reinforced concrete lining canals.

1) Concrete lining canals

This is a canal where plain concrete is laid on gradient of slopes with lining thickness of actual construction examples show 10 to 20cm. The following matters should be taken into consideration for designing the canal.

(a) When a slope form is used in concrete placement, the lining bottom shall have larger thickness than the specified value and be extruded for around 10 cm toward the base slab portion. When a slip form is used, the lining bottom is sometimes provided with a shape of circular arc.

(b) In cases where the wall height is about 2.5 m or more, contraction joints shall be provided at the position of 1/3 wall height from the base slab,

(c) Distance of contraction joints in the longitudinal direction shall be 3 to 4 m, and when slope length is especially large, closed joints or the like are required to provide in the transversal direction with an appropriate distance,

(d) Since thin linings have limitations in capability for water sealing, it is necessary in cases of sandy ground to take measures for water sealing by using impermeable materials.

2) Reinforced concrete lining canals

This is such a canal that a concrete lining canal has its lining thickness added where minimum reinforcing bars are arranged. This type is given objectives for increasing resistance against uplift pressure to the lined canals, strengthening durability for wear, and preventing most cracks as possible caused by differential settlement or temperature changes, etc. Other than these, there is a type using panels instead of cast-in-place concrete. As a reference, many examples show that construction zones of reinforced concrete lining canals are usually about 20m (about 5 panels) upstream side and about 30m (about 8 panels) downstream side of water level control weirs, about 8m (about 2 panels) at connection parts with other kinds of work (flumes, tunnels, culverts, siphons, etc.) and about 4 m (about 1 panel) of each side of upstream and downstream of division works.

It is necessary to consider the following matters for designing reinforced concrete lining canals:

(a) Descriptions on the inside cross-sections of canals are the same as of concrete lining canals.

(b) Reinforcing bars of side slope lining portions and base slab portions in the transversal direction of the canals shall be connected, but reinforcing bars in each panel in the longitudinal direction shall be independent and shall not be connected.

(3) Basement of linings

1) Banking

For basement of banking parts, surface soils shall be peeled away, garbage or the like shall be removed, and proper materials shall be filled back, so that the linings and the filling materials should come into intimate contact with each other.

2) Cutting

In cases where some or all parts of the lining canal is placed after excavating the foundation ground, the ground shall be properly treated through a judgment whether the ground is appropriate or inappropriate. In cases of unsuitable soil such as where slopes are not stable, designs shall be performed through overall discussions on necessity of replacing banking, conditions of groundwater, range of the unsuitable ground, and presence/absence of banking materials, etc.

(4) Cross-sections of canals

The bottom width/depth ratio shall be determined through considering hydraulic characteristics and cost efficiency. Many of actual accomplishments so far take the values of the bottom width/depth ratio of around 1:1 to 2:1. In general, when a large bottom width comparing to water depth is taken, it makes mechanical earth works easy, and lowered slopes enables to implement a cost efficient construction. In addition, slope stability is improved since slope length is reduced. However, too large bottom width may sometimes result in constraints from the canal site width or facilities of others, etc.

(5) Gradient of slope

Gradient of slope of concrete lining canals shall be determined through considering soil character, canal size, construction method, maintenance, etc. In general, the values of grade fall into a range of 1:1 to 1:1.5.

1) Stability of concrete lining canals depend on stability of side slopes of the canals performed by earth works. The gradient of slope shall be determined considering nature of soil, canal size, construction method, extent of compaction, distinction of cut or fill, slope height, rapid draw down of water level, etc., so that earth pressure, which can lessen the slope stability should not act on.

2) The following constraints can be named from an aspect of construction works. In cases of long and wide canals where traveling forms such as slope forms, slip forms or others are used, slump of concrete shall be appropriately selected, and gradient of slopes shall be thus determined that concrete slopes can be stable after placement.

(6) Thickness of linings

Thickness of linings shall be determined through studying the canal size, purposes, degree of importance, and conditions of future maintenance, etc. Generally, the value shall be around 10 cm as a standard. In cases of including reinforced concrete lining canals or cold climates, or where canal size is extraordinarily large, it shall be studied to increase the thickness, or others.

(7) Weep holes and underdrains

Since canals with thin linings are structurally vulnerable to uplift pressure, studies shall be pursued not only for the ordinary groundwater level but also on countermeasures against floating up by raise of groundwater or the like due to local severe rain, etc. In this connection, decompression of uplift pressure shall be in principle performed by the weep hole method or the underdrain method. However, these works are sometimes deteriorated with time, and therefore it is desirable to give sufficient allowance of safety in planning.

3.4.4 Unlined Canals

Unlined canals consist of unlined canals and protected canals. The former are canals where the natural ground is simply excavated or dikes are simply filled up on the natural ground, and the latter are canals where inner flow portions are protected by turf, stabilizer or granular fill, etc.

(1) Outline of applications

Unlined canals are generally applied to drainage canals or the like where it is not necessary to consider leakage prevention. It is favorable to make up a safe and cost efficient design through sufficient studies on alignment of the canal, longitudinal slope, cross-section and the like, so that the canal should be a structure where mechanical stability is secured and where scouring and erosion are not developed by water flow.

(2) Design of unlined canals

1) Alignment planning

It is desirable for the alignment to be as close as possible to a straight line, and studies shall be carried out with the aim of providing with a minimum number of curved sections. When a curved section is introduced due to absolute necessity, the section shall be curved in a most gentle bend possible so that energy by stream should not concentrate at a single position. Alignment of curves shall be determined referring to the following values, in response to the canal size, etc. For further details, see "3.1.7 Particulars to be considered in Selection of Canal Route and Structures"

(a) Alignment of curves

Maximum degree of curvature θ = around 60 degrees

Minimum radius of curve R = around 10B

Where B is Canal width (m)

(b) In cases where curves are adopted, continuous curves shall be avoided and a straight section with length of six times or more canal width shall be placed in between curved sections.

2) Planning for longitudinal features

Planning for longitudinal features shall be decided based on soil properties composing the canal, velocity or the like. It shall be ascertained that these values are not different from those of nearby similar canals. When the values are extensively different, they shall be appropriately revised considering distribution of river discharge or the like of the nearby similar canals. In cases of repair works for present state drainage canals or the like, longitudinal slopes of the canals are often determined basing on the longitudinal slope of the present state canals.

3) Planning for bed elevation of drainage canals

In planning for bed elevation of drainage canals, the lower elevation than the ground level in question is planned, the more capability of the canal is secured. On the other hand, the higher construction cost or the like is required. Therefore, bed elevations shall be determined considering depth of buried culverts for draining off the groundwater, access grade of field drainage, and estimated settlement of the base plane, etc.

4) Cross-sections of canals

Cross-sections of canals differ depending on the respective canal size, but generally in cases of canals where quantity of flow is small, bottom width/depth ratio of around 2:1 is adopted. Where quantity of flow is large, width/depth ratio of up to around 8:1 is applied. Inner gradient of slopes of canals often takes values of around 1:1.5 to 1:2.5, and in cases where soil character is especially of good quality and the canal size is small, inner gradient of slopes shall be able to be steeper within a range where no trouble occurs on the slope stability. In such cases, inner gradient of slopes are sometimes as steep as around 1:0.8 to 1:1.1.

On the other hand, in cases of soft ground or large-sized canals, it is often difficult to maintain slope stability, and careful studies are required for adopting gentler gradient of slopes or other bank protection methods, after grasping accurate geological features by soil tests. Canal size, purposes, site conditions or the like shall be considered for determining cross-sections of canals including gradient of slope of the canal. Table 3.38 shows general values for gradient of slope by soil character. Studies on slope stability shall also be carried out.

Table 3.38 Gradient of slopes of canals

Nature of soil	Gradient of slope
Hard rock	1 : 0.30
Weathered rock with cracks	1 : 0.50
Hard plane of clayey gravelly soil	1 : 0.50 - 1 : 1.0
Consolidated gravelly clay	1 : 1.0 - 1 : 1.5
Sand-mixed clayey loam	1 : 1.5 - 1 : 2.0
Gravel-mixed sandy loam	1 : 2.0
Sandy loam, soft clay	1 : 3.0

5) Design velocity

Velocity is decided as a consequent of longitudinal slopes and cross-sectional shapes of canals, and it is important to check whether the velocity does not cause serious scouring against canals or whether mud does not accumulate.

6) Cross-sections of watercourse and longitudinal slopes

It is desirable for cross-sections of watercourse (width, depth, cross-sectional area of flow) and longitudinal slopes to have mostly certain continuity from downstream to upstream direction. Also it shall be avoided to provide narrow sections where cross-sectional area of flow is significantly narrowed comparing with neighboring sections ahead and behind, or where the water level is swelled up.

7) Revetment planning

It shall be studied to perform linings or reinforcement for revetments of canals considering nature of soil, safety, maintenance, etc. In cases of drainage canals, banks are sometimes protected or reinforced by concrete retaining walls, concrete block masonry works, prefabricated concrete products, sheet-piles, linings or the like, considering importance of the canal, ease of maintenance works, and advantages of reducing cross-sectional area brought about by making canals compound cross-sectional, etc. In such cases, ranges of bank protection are usually and often set as high as water levels corresponding to one-year or two-year probable discharge. In cases of unlined canals, economy, effects, shapes of watercourse, placement of structures or others shall be studied, and sometimes partial revetment are required for water colliding areas or neighboring portions of structures, etc.

3.4.5 Drops

(1) General description

In the design of the canal system, when an excess head is present in spite of the appropriate distribution of the longitudinal slope of the canal and the appropriate selection of route, drops are installed in the canal. It adjusts the head, ensure the safety of the canal, and make the best use of functions of the entire canal system.

Generally, locations and types of drops are selected according to topographic features and other locational conditions. It is desirable that drops be stable and do not inhibit functions of the canal and be planned and designed by fully considering the economic efficiency of the entire canal system. A drop is a structural designed to adjust the heads of streams by means of steps and an energy dissipater. In some cases, drops may vibrate due to impact resulting from the adjustment of high heads; therefore, bearing capacity to withstand the impact as well as safety must be ensured. In addition, when drops are planned in the urban area and its vicinity, noise, vibration, and splash must be considered and studied.

(2) The type of drop works

The water-cushion type drop works is one of the types of drop works and consists of an inlet canal, which is normally a transition section from an upstream canal, outfall, water cushion section for energy dissipation, and an outlet canal for a downstream canal. In this type of drop works, vertical steps are made just below the outfall and energy is dissipated by the impact resulting from falling water clashing into a water cushion and disturbance in the water cushion. When vibration, noise or splash does not have limitation, this type can be adopted.

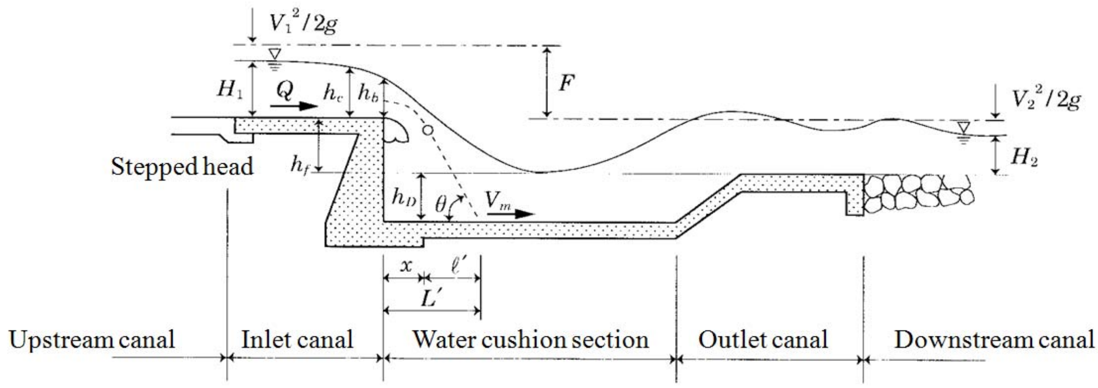


Fig. 3.39 Example of water cushion type drop works

(3) Design

1) General description

(a) When designing drop works, it is necessary to consider the prevention of inconvenient swell-head or drop down into the upstream canal as well as the prevention of disturbed flow caused by insufficient energy dissipation in the downstream canal.

(b) When installing drop works in an irrigation canal, a complete drop should be designed since design discharge has been determined. When incomplete drop or submerged flow occurs due to the influence of swell-head backwater in the escape channel at the end stream portion of the drainage canal and dual-purpose canals, design must be carried out to cope with stationary waves appearing in the downstream canal.

(c) The water level of the boundary area between an area with complete drop and an area with incomplete drop can be estimated by Equation (F. 3.71).

$$\left(\frac{h_2}{h_c}\right)^2 - \left(\frac{h_f}{h_c}\right)^2 = \frac{B_1}{B_2}\left\{3-2\left(\frac{B_1}{B_2}\right)\left(\frac{h_c}{h_2}\right)\right\} \quad \text{......................(F. 3.71)}$$

Where h_2: Water depth in the cross-section of the downstream canal (m)

h_c: Critical depth in the cross-section of the upstream canal (m)

h_f: Step difference between upstream and downstream canals (m)

B_1: Width of upstream canal (m)

B_2: Width of downstream canal (m)

To obtain a value of h_2 by using Equation (F. 3.71), a provisional calculation method for estimating h_2 is used. Calculation is easy by estimating (h_f + h_c) as first h_2.

(d) When the amount of discharge is not equal to a design discharge, incomplete overflow occurs to the drop works, generating wavy flow and decreasing the water level of upstream and downstream canals. Consequently, functions of the division works deteriorate causing unfavorable distribution of river discharges to occur in some cases. Accordingly, it is desirable that distribution of river discharges be researched in regards to the amount of discharge not equal to a design discharge; for example, most frequent discharge in the irrigation canal and the discharge for low-level revetment in the drainage canal.

(e) In the drop works, negative pressure is generated on the back side of the falling water vein, causing the water vein to vibrate. And, because low-frequency noise problems may occur, it is necessary to install a pipe to supply air to the back side of the falling water vein or a spoiler to cut the water vein.

2) Upstream canal

The necessary length of the upstream canal of the drop works must be ensured so that the side slopes and the bottom of the canal will not be eroded or scoured due to the increase in flow velocity resulting from the lowering of water surface. It is desirable that the length of the upstream canal be obtained by backwater calculation, however, usually it is obtained by using Equation (F. 3.72), Equation (F. 3.73), and Equation (F. 3.74). With regard to a length longer than the above value, necessary measures should be taken.

In the case of $q \leq 2$ $\quad L = 1.2 + \frac{3}{2}\sqrt{Q}$ ....(F. 3.72)

In the case of $q > 2$ $\quad L = 2.1 + \frac{3}{2}\sqrt{Q}$ ....(F. 3.73)

or, $L = 4z$ ....(F. 3.74)

Where

L: Length of upstream canal (m)
Q: Discharge (m^3/S)
q: Discharge per unit width ($m^3 \cdot s^{-1} \cdot m^{-1}$)
z: Water level difference between upstream and downstream (m)

In the case of q>2, the value obtained by Equation (F. 3.73) or the value obtained by Equation (F. 3.74), whichever is larger is used. When the upstream canal is a three-side concrete lined canal, it is not necessary to consider the upstream canal.

3) Outfall

(a) Shape

The cross-section of the outfall sally be basically rectangular in order to equalize the flow condition in the canal's cross-section to perfect the functions of the energy dissipater. However, in the drainage canal, there are many examples in which the cross-section of the outfall is trapezoidal as is the same as in the upstream canal. Furthermore, with respect to the flow just below the outfall, sufficient air should be supplied to the bottom surface of the falling water vein. In order to prevent hydraulic influences on the upstream canal associated with lowering of water surface at the drop works outfall, measures should be taken for the drop works outfall; for example, narrowing the width, increasing the bed height, or narrowing the width and increasing the bed height. From the view point of safety of canal's functions, when planning to install drop works in the drainage canal, it is desirable that the cross-section of the outfall be the same shape as that of the upstream canal instead of narrowing the width of the outfall or raising the height of the head.

(b) Value of the water depth at the outfall

If the head is raised at least 0.3 times the critical depth, the water depth h_b at the outfall is supposed to be equal to the critical depth h_c. Furthermore, at the outfall having a rectangular cross-section in which the head is not raised, h_b/h_c is considered to be nearly 0.72. Furthermore, at the outfall having a trapezoidal cross-section in which the head is not raised, the ratio h_b/h_c, of outfall water depth h_b to critical depth h_c is considered to be nearly 0.72 from a practical view point.

(c) Critical depth

In a rectangular section, the critical depth can be expressed by using Equation (F. 3.13). Refer to 3.2.4(2) for the detail.

4) Shape of falling water vein and energy dissipater

The design method is as described below. A water-cushion type energy dissipater is designed such that a water vein comes in contact with a downstream water-cushion and disturbed, thereby dissipating energy. Generally, the water-cushion type energy dissipater is adopted when sufficient depth of downstream water is available with respect to the depth of hydraulic jump of the virtual falling water vein.

(a) Shape of water vein (hydraulic characteristics until immediately before encounter)

The track of falling water vein, angle of inclination, velocity, and thickness of water vein can be expressed by using Equation (F. 3.75) to Equation (F. 3.80).

a) Track of the center of water vein

Without afflux $x/H = 1.477\ \{(y/H) + 0.242\}^{0.567}$ ....(F. 3.75)

With afflux $x/H = 1.155\ \{(y/H) + 0.333\}^{0.500}$ ....(F. 3.76)

However, "with afflux" means the condition in which the head is raised about 0.3 times the critical depth.

b) Inclination angle of water vein

Without afflux $\tan\theta = 0.866\ (x/H)^{0.763}$ ....(F. 3.77)

With afflux $\tan\theta = 1.500\ (x/H)$ ....(F. 3.78)

c) Velocity at the center of water vein

$$V = \sqrt{2g \cdot z} \quad \text{.......(F. 3.79)}$$

d) Thickness of water vein

$$d = Q / (B \cdot V) \quad \text{.......(F. 3.80)}$$

Where

H: Specific energy at the critical depth of upstream canal (however, H=1.5 h_c) (m)
h_c: Critical depth of upstream canal (outfall) (m)
x: Horizontal distance measured downward from the outfall's downstream end (m)
y: Vertical distance measured downward from the outfall's downstream end (m)
θ: Inclination angle of water vein's center line at arbitrary point (x, y) (°)
V: Velocity (m/s)
g: Gravity acceleration 9.8 (m/s^2)
z: Head from energy line to the center of water vein (m)
d: Thickness of falling water vein (m)
Q: Discharge (m^3/s)
B: Width of outfall (m)

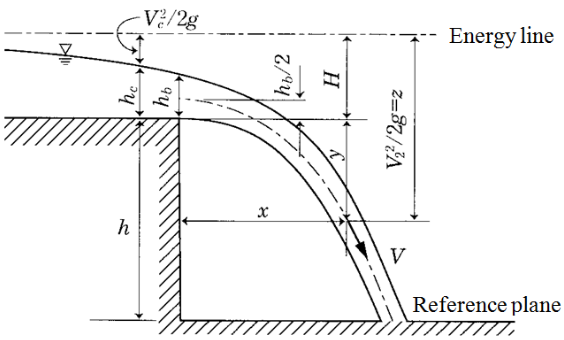


Fig. 3.40 Explanatory drawing of falling water vein (free drop)

(b) Flow velocity of downward water vein in the water cushion

The relation among flow velocity V_{max} at an arbitrary point in the penetration direction which is on the extension of the center entry angle of falling water vein, water vein center penetration velocity V, penetration distance S, and thickness d of water vein at the penetration point is expressed by Equation (F. 3.81) and Equation (F. 3.82).

In the case of $S \leq 5.82d$ $\quad V_{max} = V$ ....(F. 3.81)

In the case of $S > 5.82d$ $\quad V_{max} / V = 2.41 / \sqrt{S / d}$ ....(F. 3.82)

Where

V_{max}: Flow velocity at an arbitrary point in the penetration direction which is on the extension of the center entry angle of falling water vein (m/s)
V : Water vein center penetration velocity (m/s)
S : Penetration distance (m)
D : Thickness d of water vein at the penetration point (m)

(c) Length of the water cushion (L_0)

Length L_0 of the water cushion is based on the horizontal distance L which is from the outfall section of the falling water vein to the location at which the central portion of water vein comes in contact with the bottom of the water cushion. And, when the width B_0 of the water cushion is wider than the width of the upstream canal, $L_0 \geq 2.5L$; and when the width B_0 of the water cushion is equal to the width of the downstream canal, $L_0 \geq 3.0L$.

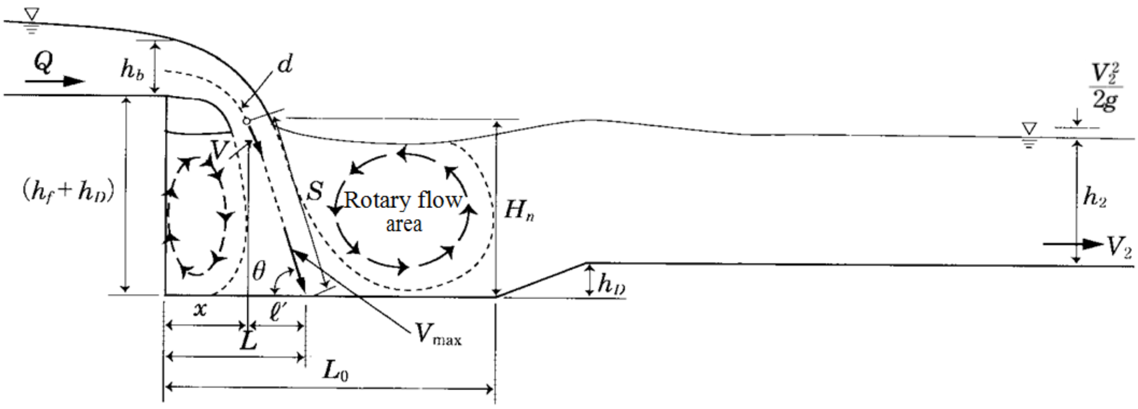


Fig. 3.41 Explanatory drawing of falling water diffusion in the water cushion

To provide appropriate depth h_D and width B_0 of the water cushion and sufficiently dissipate energy in the water cushion, Equation (F. 3.83) for the hydrostatic pressure P at the end of the water cushion must be satisfied. However, the width of the water cushion shall be within twice the outfall. If Equation (F. 3.83) is not satisfied, values of depth h_D and width B_0 of the water cushion should be revised, and calculation should be repeated.

$$P > 3M \quad \text{.......(F. 3.83)}$$

Where

P: Hydrostatic pressure at the end of the water cushion ($=\gamma_w \cdot B_0 \cdot H_n^2 / 2$) (kN)
M: Percentage (Force) of momentum change per unit time ($\gamma_w \cdot Q \cdot V_m / g$) (kN) at a location at which a falling water vein reaches the bottom of the water cushion
γ_w: Unit volume weight of water (kN/m^3)

Q: Discharge (m^3/s)

H_n: Water depth at the end of the water cushion ($\approx h_D+h_2+V_2^2/2g$)

V_m: Velocity at the time a falling water vein reaches the bottom of the water cushion (m/s)

g: Gravity acceleration 9.8 (m/s^2)

5) Ruffle in the case of incomplete drop

(a) Attenuation of center velocity in the downstream canal

In the case of incomplete drop, the maximum flow velocity exists near the water surface of the downstream canal; therefore, the water cushion does not influence dissipation effect. Maximum velocity hardly attenuates within the range 8 to 10 times the water depth at the outfall and shows the value equal to the mean velocity at the outfall. For this reason, in the drop with a downstream canal, it is desirable that the start point of the canal be located at the location at least 8 times the critical depth h_c of the upstream canal downward from the outfall intended for design discharge flow. The splay angle of the downstream canal is equal to or less than 12°30'.

(b) Ruffle in the downstream canal

Stationary waves occur in the downstream canal in the state of incomplete drop, and the wave height gradually smaller than that of the first wave. The approximate value of ratio W_{a1}/h_c between the wave height W_{a1} of the first wave and the critical depth h_c of the upstream canal is 0.6. Accordingly, it can be estimated that the water depth of the downstream canal is higher by 0.3 which is a half of the value 0.6. Furthermore, when the Froude number of the downstream canal is equal to or less than 0.2, distance L at which ruffle finishes is nearly 30 times the critical depth of the upstream canal. Thus, in cases in which there is a possibility that side slopes of the downstream canal connected to the drop may be scoured, installation of revetment must be considered in the range where L/h_c is up to 30.

Where

Q: Discharge (m^3/s)

h_f: Height of head (m)

h_c: Critical depth (m)

H_2: Water depth of downstream canal (m)

W_{a1}: Height of first wave (m)

W_{a2}: Height of second wave (m)

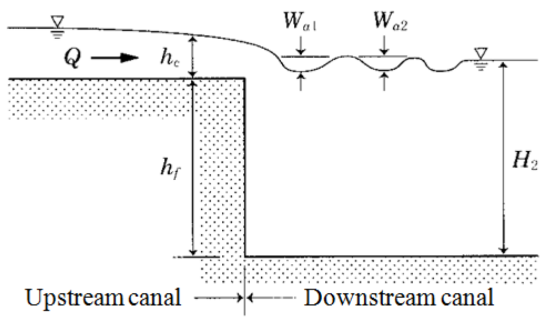


Fig. 3.42

Explanatory drawing of height of waves near the outfall

6) Downstream canal

Specifications of the shape of the transition section connecting the energy dissipater to the downstream canal must be determined so that flow velocity does not change rapidly and irregular flows and waves will not occur on the downstream water surface. It is desirable that the splay angle be kept 12°30' on the water surface and the gradient of the bottom of the canal be more moderate than 1:4. Furthermore, in order to cope with remaining turbulence of flow and vortex, in some cases, protective structure is additionally installed downstream the approach canal. When the canal connected to downstream of the drop works is an unlined canal, it is desirable that the length of the protective structure in the downstream canal be as long as the length of the water cushion.

7) Design of construction joint

In the event sidewalls and the bottom slab are moved and deformed due to the joint, careful attention should be taken because the structures may block the stream thereby adversely affecting the drop body. Furthermore, it is necessary to take measures to prevent such displacement when designing the drop works. Moreover, as a reference, Fig. 3.43 shows the joint structure commonly used. As shown in Fig. 3.43 (a), the bottom slab is fixed to a cut-off wall on the upstream side. Furthermore, structures of the vertical step wall of the small-scale drop works and the bottom slab of the upstream transition section shall be as shown in Fig. 3.43 (b), and the bottom slab of the transition section shall be a simply supporting structure. This is because of the prevention of cracking on the wall due to settlement since this section becomes a banking portion resulting from excavation. When the thickness of pavement of the canal is thin, the projecting portion of downstream concrete is inserted under the upstream end as shown in Fig. 3.43 (c) so as to prevent the downstream block from being lifted by the inflow. Quality of concrete poured on the slope must be fully controlled so that expected effects can be obtained as designed.

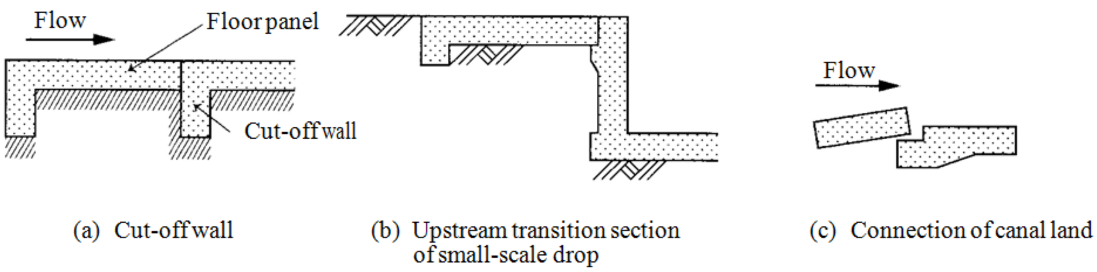


Fig. 3.43 Joint structure

3.4.6 Division Boxes

(1) General description

When installing division boxes, it is necessary to plan the division boxes with prescribed functions based on the irrigation network studied in designing the canal system, scrap-and-build plan for division boxes, and the diversion plan and its arrangement of route system, which consider the local irrigation custom and the will of the community. Accordingly, before designing a facility, design consideration points based on the canal system must be confirmed; for example, water supply plan, water management system, type of division box considering the canal type, scale, and the number of facilities. To select the installation location, the following matters should be considered from the viewpoint of maintenance, management and structural design.

1) The location shall be on the good ground,
2) The location shall be near the irrigated area and convenient for the maintenance and management of the division boxes,
3) High banking or deep cutting shall be avoided as much as possible,
4) The location where there is no possibility of the occurrence of disaster,
5) The location at which construction can be executed easily,
6) The location where water flow is stabilized,
7) The location where necessary head can be ensured.

(2) Consideration points for designing division boxes

When designing division boxes, it is desirable that the following matters be considered.

1) The structure shall be rigid, accurate, and durable,
2) The costs of facility, maintenance and management shall be economical,
3) Division boxes shall not cause hydraulic state of upstream and downstream to change significantly,
4) Although division boxes are designed based on the design discharge, the amount of discharge other than the design discharge, such as most frequent discharge, minimum discharge, etc., shall be considered according to the functions and purpose of each division box.
5) Head loss for diversion shall be minimized and easily regulated from the viewpoint of maintenance and management.

6) Functions of pressure regulation, elimination of air, settling basin, and spillways can be provided for division boxes with free surface. Therefore, head distribution and dual-purpose function shall be considered,
7) Depending on the selection of valves used for division boxes, large water hammer pressure occurs causing the barrel to break. Accordingly, valve opening/closing velocity shall be also considered,
8) Scale of the division box stand shall be determined by considering water surface vibration by surging and ensuring the seal of pipelines.

(3) Design

1) Water-gate type division boxes

Coefficients of discharge of water gate outflow differ significantly according to the type of water gate. Furthermore, according to the condition of flow, outflow from the water gate can be classified into two types as shown in Fig. 3.44: (a) and (b). These are (a) free outflow (an outflow water vein from the water gate is a supercritical flow and continues into downstream flow by hydraulic jump); and (b) submerged outflow (an outflow water vein is submerged under the downstream water surface). Various equations have been proposed to obtain the amount of discharge from the water gate. However, this guideline uses Equation (F. 3.84) which can be applied to both the free outflow and the submerged outflow.

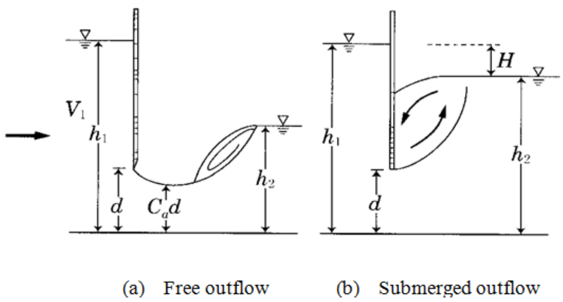


Fig. 3.44 Water gate

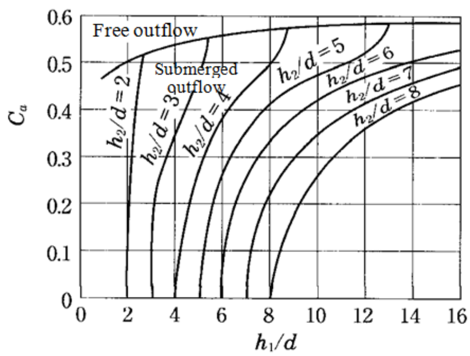


Fig. 3.45 Coefficient of discharge Ca at water gate

Equation (F. 3.84) is Henry's experimental equation at the flap gate, and the relation between coefficient of discharge Ca and h_1/d, h_2/d is as shown in Fig. 3.45.

$$Q = C_a \cdot b \cdot d \sqrt{2g \cdot h_1} \quad \text{(F. 3.84)}$$

Where Q: Discharge (m3/s)

C_a: Coefficient of discharge obtained from Fig. 3.45

b, d: Inner space width and opening height of gate (m)

h_1: Water depth upstream of gate (m)

h_2: Water depth downstream of gate (m)

C_c: Coefficients of contraction

On the other hand, when obtaining an opening of gate or water depth upstream or downstream of the gate, Equation (F. 3.84) is not convenient; therefore, in those cases, Equation (F. 3.85) and Equation (F. 3.86) can be used. Moreover, because results vary greatly due to coefficients of discharge in Equation (F. 3.85) and Equation (F. 3.86), in some cases, revision may be necessary by obtaining actual measurements after completion.

In the case of free outflow

$$Q = C_1 \cdot b \cdot d\sqrt{2g\,(h_1 - d)} \quad \text{.......................................} \quad \text{(F. 3.85)}$$

In the case of submerged outflow

$$Q = C_2 \cdot b \cdot d\sqrt{2g\,(h_1 - h_2)} \quad \text{.......................................} \quad \text{(F. 3.86)}$$

Where Q: Discharge (m^3/s)

C_1, C_2: Coefficient of discharge; between 0.62 and 0.66 when $h_1/d > 2.5$ according to Yokota

b, d: Inner space width and opening height of gate (m)

h_1: Water depth of main canal (upstream of gate) (m)

h_2: Water depth of diversion channel (downstream of gate) (m)

2) Sluice-pipe type division works

When water flow is diverted from the main canal through pipe, discharge can be obtained by Equation (F. 3.87).

$$Q = A\sqrt{\frac{2gh}{f_e + f_0 + f \cdot L/D}} \quad \text{.....................................} \text{(F. 3.87)}$$

Where Q: Discharge (m^3/s)

A: Cross-sectional area of flow of diversion pipe (m^2)

f_e : Inflow loss coefficient (Table 3.39)

f : Pipe's friction loss coefficient $f = 124.5n^2 / D^{1/3}$

n: Coefficient of roughness of diversion pipe

L: Length of diversion pipe (m)

D: Diameter of diversion pipe (m)
h: Difference between main canal's water level and sluice-pipe outlet's water level (m)
g: Gravity acceleration 9.8 (m/s^2)
f_o: Outflow loss coefficient (f_0 = 1.0)

As shown in Fig. 3.46, when a diversion pipe is directly mounted to the sidewall of the main canal to divert water to the diversion channel, inflow loss coefficient fe is as shown in Table 3.39.

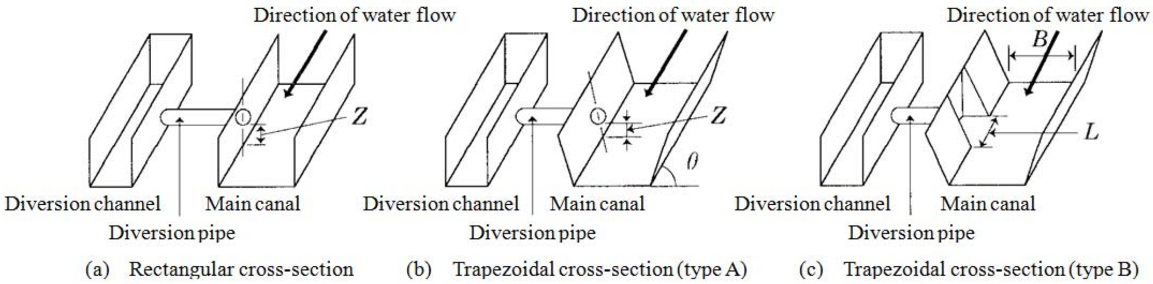


Fig. 3.46 Shape of sluice-pipe type division work

Table 3.39 Loss coefficient (fe) of sluice-pipe type division work due to inflow

Shape of main canal	Loss coefficient due to inflow (f_e)	Note	Applicable scope
(a) Rectangular cross-section (b) Trapezoidal cross-section (type A) (c) Trapezoidal cross-section (type B)	$f_e = 0.5$ $f_e = \frac{a}{F \cdot b} + 0.5$ $\begin{cases} a = \frac{0.24}{\left(\frac{D}{0.05}\right)^3} + 0.37m^3 \\ b = \frac{0.8}{m} \end{cases}$ $f_e = \frac{c}{F \cdot d} + 0.5$ $\begin{cases} c = 0.37\,m \\ d = 1.5/m^{0.5} \end{cases}$	$F = \frac{V}{\sqrt{g \cdot D}}$ V: Mean velocity in diversion pipe (m/s) g: Gravity acceleration 9.8(m/s^2) D: Inner diameter of diversion pipe (m) m: Gradient of slope of main canal (cotθ) θ is an angle (°) formed by side-wall slope and horizontal plane Z: Installation height of diversion pipe (m)	1. Discharge is one tenth of discharge of main canal or less 2. Froude number of flow in main canal is 0.54 or less. 3. Value of D is 0.05 m or more and one third of water depth H or less 4. Length L of partially recessed portion of type B canal is $2D<L<4D$ 5. Covering water depth h at diversion pipe's inlet is $1.5D<h<4D$ 6. Value of Z is 0.05 m or more. 7. Value of m is $0<m<1.0$ 8. Value of B is $3D<B<8D$

4. Check list for canal

Check list is one of the effective methods to prevent oversight and mistake and to easily understand the condition of design. At the present's situation, some of investigation and design can't be done, because of the problem of cost and equipment. These necessary matters are done during construction time. So it is not necessary to check the entire item before construction. But there are necessary item that should be checked at construction time. So check list is made separately one for before construction, another for at construction time. Format of check list is next page. When the answer is "yes", check into □. And fill the necessary information. Some of the item is not necessary according to the situation of each project.

Check list for canal before construction (1/3)

Item	Contents	Check	Remark
Survey	Topographic survey Longitudinal (canal route) survey Cross-section survey Survey for temporary works Collection of topographic map	☐ ☐ ☐ ☐ ☐	Km section
Design intake water level	There is consideration for irrigation area There is consideration for protecting flowing in sediment.	☐ ☐	
Condition of canal (foundation, side wall)	How to investigate Drilling Test pitting Observation Bearing test by standard penetration test Bearing test by Loading test Investigation on river deposit Ground water investigation	☐ ☐ ☐ ☐ ☐ ☐ ☐	
Dikes, Bridge and other structures	Data is collected	☐	
Present condition of river water utilization	Data is collected	☐	

Check list for canal before construction (2/3)

Item	Contents	Check	Remark
Investigation for construction works	Meteorology	☐	
	Surface water	☐	
	Ground water	☐	
	Construction equipment	☐	
	Construction material(cement)	☐	
	Construction material(gravel)	☐	
	Construction material(sand)	☐	
	Construction material(stone for masonry)	☐	
	Transportation	☐	
	Electric	☐	
Other	Environmental impact assessment	☐	
Type of canal	Earthen type	☐	
	Masonry type	☐	
	Lining (concrete) type	☐	
Design discharge	There is ground data and document	☐	

Check list for canal before construction (3/3)

Item	Contents	Check	Remark
Velocity	The velocity is more than minimum allowable velocity The velocity is less than maximum allowable velocity	☐ ☐	
Canal side wall stability	Overturning ok Sliding ok Settlement ok Stress ok	☐ ☐ ☐ ☐	
Gate	There is consideration for not bending There is consideration for operation by human	☐ ☐	
Wing wall	It has enough height It has enough stability	☐ ☐	
Settling basin	The design is done by guideline	☐	
Spill way		☐	

5. EXAMPLE OF DESIGN FOR CANAL AND RELATED STRUCTURES

This Chapter shows an example of crucial data and calculation for basic design of canal and related structures. In case of actual design, designer must follow the contents of this Chapter.

5.1 Design of Canal

5.1.1 Basic Data for Design of Canal

(1) Data from surveyor

- Topographic map, cross-section, longitudinal-section (canal profile)
- The highest elevation of the irrigation area (starting point of irrigation area) = EL.2420.5m
- Canal length from starting point of canal to starting point of irrigation area (the highest elevation of the irrigation area) = 1300m
  ※This data is an assumption value at first
- Canal slope = 1/1000 (Lined canal), 2/1000 (Earthen canal)
  ※This data is an assumption value at first

(2) Data from hydrologist

- The amount of usable water for irrigation = 63 l/s
  (Comparison with Agronomist’s data)

(3)Data from Agronomist

- The crop water requirement = 63 l/s
  (Comparison with Hydrologist’s data)

(4) Data from geologist

- The geological condition on the canal route

Chain-age	Geology	Topography	Expected risk	Minimum Depth from surface to hard rock
0+00 to 0+100	Porous soil with soft rock and boulder stones	Flat and stable slope	Water loss due to leakage	3.0m
0+100 to 0+200	Slightly soft rock	Stable slope But at 0+100 need deep cut	Difficulty of excavation this rock	1.2m

0+200 to 0+325	Soil underlain by highly weathered rock	Stable slope and Flat	Water loss due to leakage problem	1.0m
0+325 to 0+381	Hard Ignimbrite rock	Steep slope	Seepage and canal breach if earthen canal assumed	0.3m
0+381 to 0+500	Partly soil and Partly soft rock	steep slope	Piping failure at soil and rock interface if Earthen canal assumed	Unmeasured
0+500 to 0+700	Thick soil	Flat and Stable slope	Seepage problem if Earthen canal assumed	Unmeasured
0+700 to 1+000	partly soft rock and thick soil partly Highly Weathered Tuff and Ash rock	Steep Slope	Leakage problem due to weathered rock	0.2m
1+000 to 1+280	Porous soil and partly weathered Tuff and ash	Steep slope	Leakage problem if canal is Earthen	Unmeasured
1+280 to 2+400 onset with command	Water tight dark heavy clay	Stable and flat slope	No expected risk so Earthen canal is possible	Unmeasured

5.1.2 Canal Condition (Chapter3.1)

Based on 5.1.1, the canal condition will be assumed as follows;

Type of canal	Place	Length (m)	Discharge (m3/s)	Slope
Retaining wall canal (Masonry) or Berm canal	0m -1280m	1280	0.063	1/1000 = 0.001
Earthen canal (Clay soil) or Berm canal	1280m - 2400m	1120	0.063	2/1000 = 0.002

5.1.3 Canal Shape

(Chapter 3.2.2, 3.2.3, 3.3.2, 3.3.3, 3.3.4, 3.3.5, 3.4.1, 3.4.4)

(1) Hydraulic design ….. (See Chapter 3.2.2, 3.2.3)

Based on 5.1.2 and according to the Chapter 3.2.2 and 3.2.3, the allowable velocity and coefficient of roughness is as follows;

Type of canal	Place	Length (m)	Discharge (m3/s)	Slope	Minimum allowable velocity (m/s)	Maximum allowable velocity (m/s)	Coefficient of Roughness
Retaining wall canal (Masonry)	0m - 1280m	1280	0.063	1/1000 =0.001	0.45	1.5*	0.015 (Concrete (cast-in place flume, culvert, etc.))
Earthen canal (Clay soil)	1280m - 2400m	1120	0.063	1/500 =0.002	0.45	1.0 (Clay)	0.025 (Earthen canals, curved and non-uniform (No vegetation coverage))

* : Maximum allowable velocity for Retaining wall canal (Masonry) is 1.5m/s as thin concrete (approximately10cm). Because there is plastering on the surface of canal.

And it is known that the stable water surface can be expected at least if the velocity is equal to or less than approximately two-thirds of the critical velocity (Froude number: 0.54) under identical discharge condition. …… (See 3.2.2(4))

$Fr = v / \sqrt{(g \times h)} < 0.54$

(2) Cross-section form …… (See 3.4.1(1), 3.4.4(2)4))

According to Chapter 3.4.1(1), hydraulically favorable cross-section is decided by the following formula,

$$B = 2H \times \tan(\theta/2)$$

And the case of unlined canal, according to Chapter 3.4.4(2)4) Table 3.3.38, the slope of canal is decided.

Table 3.38

Nature of soil	Gradient of slope
Hard rock	1 : 0.30
Weathered rock with cracks	1 : 0.50
Hard plane of clayey gravelly soil	1 : 0.50 - 1 : 1.0
Consolidated gravelly clay	1 : 1.0 - 1 : 1.5
Sand-mixed clayey loam	1 : 1.5 - 1 : 2.0
Gravel-mixed sandy loam	1 : 2.0
Sandy loam, soft clay	1 : 3.0

(3) Canal freeboard …… (See 3.4.1(2))

1) In case of normal canal ($Fr = v/\sqrt{gh} \leq 1$)

Canal freeboard is decided by the following formula and Fig. 3.36 in 3.4.1(2),

(a) Retaining wall canals (flumes, retaining wall canals (masonry), box culverts, ready-made product canals, etc.)

$$Fb = 0.07d + \beta \times hv + hw$$

(b) Non-lining canals and lining canals by concrete

$$Fb = 0.05d + \beta \times hv + hw$$

2) In case of supercritical or rapid flow canal ($Fr = v/\sqrt{gh} > 1$)

Canal freeboard is decided by the following formula,

$$Fb = C \times Vh^{1/2}$$

$$Fb = 0.6 + 0.037 \times Vh^{1/3}$$

There is a range at the length of freeboard calculated above formula, so freeboard is decided based on the actual site condition.

(4) Calculation on canal shape

Based on (1)~(3) above, canal shape is decided.

The check points are as follows,

1) Minimum and maximum allowable velocity
2) Froude number
3) Cross-section form
4) Canal freeboard

Refer to Excel format guide 1 for the calculation.

Excel format guide 1. Canal size (non-lining and lining canal) ※In the case of normal slope canal : Fr < 1

#Please input data into yellow cell.
#The input data of green cell is the value assumed yourself. (Goal seek "By changing cell")
#Blue cell is the "set sell" for Goal seek. "to value" = Qd value

1. Consideration of B and h of canal

(1)Design discharge (Qd) =		0.063	m3/s	
(2)Coefficient of Roughness (n) =		0.025		
(3)Slope of canal (I) =		0.002		
(4)Side slope of wall (m) =	1:	1	=	0.79 rad
(5)Minimum allowable velocity (V_{min}) =		0.45	m/s	
(6)Maximum allowable velocity (V_{max}) =		1.0	m/s	

When check point ① and ② is OK, compare with canal width B and ③, and select B which is near value with ③

This value is assumption
↓ ↓Goal seek "By changing cell" ↓Goal seek ("set sell". "to value" = Qd value)

Canal width B(m)	Water depth h(m)	Water area A(m2)	Wetted Perimeter P(m)	Hydraulic radius R(m)	Velocity V(m/s)	Discharge (Goal seek) Q(m3/s)	Check points ①Velocity 0.45 <V< 1.0	②Froude number Fr	Fr<0.54	③Recommendation B=2h*tan(θ/2)	Result
0.100	0.321	0.135	1.009	0.134	0.469	0.063	OK	0.149	OK	1.547	
0.150	0.299	0.135	0.997	0.135	0.471	0.063	OK	0.160	OK	1.443	Select
0.200	0.279	0.134	0.989	0.135	0.471	0.063	OK	0.172	OK	1.344	
0.250	0.263	0.135	0.995	0.136	0.473	0.064	OK	0.183	OK	1.269	
0.300	0.246	0.134	0.995	0.135	0.470	0.063	OK	0.195	OK	1.184	
0.350	0.233	0.136	1.009	0.135	0.470	0.064	OK	0.206	OK	1.123	
0.400	0.219	0.135	1.019	0.133	0.466	0.063	OK	0.217	OK	1.054	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	

2. Freebard of canal (check point ④)

(1)Form of canal (α) =	0.05	Retaining wall canals : 0.07 (flumes, retaining wall canals, box culverts, ready-made product canals, etc.) Non-lining canals and lining canals : 0.05
(2)Conversion factor from velocity head to static head (β) =	1.0	
(3)Freeboard for water surface vibration (hw) =	0.15	

↓Goal seek "By changing cell" ↓Goal seek ("set cell") ("to value" = 1.2*Design discharge(a) value)

Calculation of freebard: Canal width B(m)	Water depth d(m)	velocity V(m/s)	Velocity head hv(m)	Freeboard Fb(m)	Necessary canal height (m)	Confirmation of the case of 1.2*Qd: 1.2*Design discharge (a) (m3/s)	Water depth d'(m)	Water area A(m2)	Wetted Perimeter P(m)	Hydraulic radius R(m)	Velocity V(m/s)	1.2*Design discharge (b) (m3/s)	Result of confirmation (compare with "Necessary canal height" and "Water depth")
0.100	0.321	0.469	0.011	0.177	0.498	0.0756	0.345	0.154	1.077	0.143	0.489	0.0752	OK
0.150	0.299	0.471	0.011	0.176	0.476	0.0756	0.324	0.153	1.065	0.144	0.491	0.0753	OK
0.200	0.279	0.471	0.011	0.175	0.454	0.0756	0.304	0.153	1.059	0.144	0.493	0.0754	OK
0.250	0.263	0.473	0.011	0.175	0.438	0.0756	0.286	0.153	1.058	0.145	0.493	0.0755	OK
0.300	0.246	0.470	0.011	0.174	0.419	0.0756	0.271	0.155	1.067	0.145	0.494	0.0765	OK
0.350	0.233	0.470	0.011	0.173	0.406	0.0756	0.256	0.155	1.073	0.144	0.492	0.0762	OK
0.400	0.219	0.466	0.011	0.172	0.391	0.0756	0.241	0.155	1.082	0.143	0.489	0.0757	OK
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Excel format guide 1. Canal size (retainig wall canal) ※In the case of normal slope canal : Fr < 1

#Please input data into yellow cell.
#The input data of green cell is the value assumed yourself. (Goal seek "By changing cell")
#Blue cell is the "set sell" for Goal seek. "to value" = Qd value

1. Consideration of B and h of canal

(1)Design discharge (Qd) = 0.063 m3/s
(2)Coefficient of Roughness (n) = 0.015
(3)Slope ofcanal (I) = 0.001
(4)Side slope of wall (m) = 1: 0 = 1.57 rad
(5)Minimum allowable velocity (V_{min}) = 0.45 m/s
(6)Maximum allowable velocity (V_{max}) = 1.5 m/s

When check point ① and ② is OK, compare with canal width B and ③, and select B which is near value with ③

This value is assumption
↓ ↓Goal seek "By changing cell" ↓Goal seek ("set cell". "to value" = Qd value) ↓

Canal width B(m)	Water depth h(m)	Water area A(m2)	Wetted Perimeter P(m)	Hydraulic radius R(m)	Velocity V(m/s)	Discharge (Goal seek) Q(m3/s)	Check points ①Velocity 0.45 <V< 1.5	②Froude number Fr	Fr<0.54	③Recommendation B=2h*tan(θ/2)	Result
0.300	0.431	0.129	1.163	0.111	0.488	0.063	OK	0.115	OK	0.862	
0.350	0.358	0.125	1.067	0.118	0.506	0.063	OK	0.144	OK	0.716	
0.400	0.307	0.123	1.013	0.121	0.516	0.063	OK	0.172	OK	0.613	
0.450	0.270	0.121	0.989	0.123	0.520	0.063	OK	0.197	OK	0.539	
0.500	0.243	0.121	0.986	0.123	0.522	0.063	OK	0.219	OK	0.485	Select
0.550	0.221	0.121	0.991	0.122	0.520	0.063	OK	0.240	OK	0.441	
0.600	0.202	0.121	1.004	0.121	0.515	0.062	OK	0.260	OK	0.404	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	

2. Freebard of canal (check point ④)

(1)Form of canal (α) = 0.07 Retaining wall canals : 0.07
(flumes, retaining wall canals, box culverts, ready-made product canals, etc.)
Non-lining canals and lining canals : 0.05

(2)Conversion factor from velocity head to static head (β) = 1.0
(3)Freeboard for water surface vibration (hw) = 0.15

↓Goal seek "By changing cell" ↓Goal seek ("set cell")
("to value" = 1.2*Design discharge(a) value)

Calculation of freebard						Confirmation of the case of 1.2*Qd							Result of confirmation (compare with "Necessary canal height" and "Water depth")
Canal width B(m)	Water depth d(m)	velocity V(m/s)	Velocity head hv(m)	Freeboard Fb(m)	Necessary canal height (m)	1.2*Design discharge (a) (m3/s)	Water depth d'(m)	Water area A(m2)	Wetted Perimeter P(m)	Hydraulic radius R(m)	Velocity V(m/s)	1.2*Design discharge (b) (m3/s)	
0.300	0.431	0.488	0.012	0.192	0.624	0.0756	0.505	0.152	1.310	0.116	0.500	0.0759	OK
0.350	0.358	0.506	0.013	0.188	0.547	0.0756	0.414	0.145	1.179	0.123	0.522	0.0757	OK
0.400	0.307	0.516	0.014	0.185	0.492	0.0756	0.355	0.142	1.110	0.128	0.535	0.0761	OK
0.450	0.270	0.520	0.014	0.183	0.452	0.0756	0.311	0.140	1.072	0.131	0.542	0.0759	OK
0.500	0.243	0.522	0.014	0.181	0.424	0.0756	0.278	0.139	1.055	0.132	0.545	0.0757	OK
0.550	0.221	0.520	0.014	0.179	0.400	0.0756	0.252	0.139	1.054	0.132	0.545	0.0756	OK
0.600	0.202	0.515	0.014	0.178	0.380	0.0756	0.232	0.139	1.064	0.131	0.543	0.0757	OK
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.0756		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Excel format guide 1. Canal size (supercritical or rapid flow canal) ※In the case of steep slope canal : Fr > 1

#Please input data into yellow cell.
#The input data of green cell is the value assumed yourself. (Goal seek "By changing cell")
#Blue cell is the "set sell" for Goal seek. "to value" = Qd value

1. Consideration of B and h of canal

(1)Design discharge (Qd) =		0.063	m3/s		
(2)Coefficient of Roughness (n) =		0.015			
(3)Slope ofcanal (I) =		0.07			
(4)Side slope of wall (m) =	1:	1	=	0.79 rad	
(5)Minimum allowable velocity (V_{min}) =		0.45	m/s		
(6)Maximum allowable velocity (V_{max}) =		2.25	m/s		(t=14cm)

When check point ① is OK and ② is greater than 1, compare with canal width B and ③, and select B which is near value with ③

This value is assumption
↓ ↓Goal seek "By changing cell" ↓Goal seek ("set cell". "to value" = Qd)

Canal width B(m)	Water depth h(m)	Water area A(m2)	Wetted Perimeter P(m)	Hydraulic radius R(m)	Velocity V(m/s)	Discharge (Goal seek) Q(m3/s)	Check points ①Velocity 0.45 <V< 2.25	②Froude number Fr	③Recommendation B=2h*tan(θ/2)	Result
0.400	0.059	0.027	0.567	0.048	2.323	0.063	NG	4.015	0.284	
0.450	0.055	0.028	0.606	0.046	2.265	0.063	NG	4.187	0.266	
0.500	0.052	0.029	0.647	0.044	2.208	0.063	OK	4.341	0.250	Select
0.550	0.049	0.029	0.689	0.043	2.153	0.063	OK	4.481	0.236	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	
		0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.000	

2. Freebard of canal (ckeck point④)

①Cofficient (C) = 0.13 Rectangular canal : 0.1
Trapezoidal canal : 0.13

Canal width B(m)	Water depth h(m)	velocity V(m/s)	Freeboard $Fb=CVh^{1/2}$ Fb(m)	$Fb=0.6+0.037Vh^{1/3}$ Fb(m)
0.400	0.059	2.323	0.073	0.633
0.450	0.055	2.265	0.069	0.632
0.500	0.052	2.208	0.065	0.630
0.550	0.049	2.153	0.062	0.629
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!
0.000	0.000	#DIV/0!	#DIV/0!	#DIV/0!

The result of calculation and check above four points, the proper canal cross-section is below.

Type of canal	Discharge (m3/s)	Slope	Coefficient of Roughness	Canal width (m)	Water depth (m)	Side slope	Velocity (m/s)	Free board (m)	Necessary canal height (m)	Design canal height (m)	Hard coring (m)	Lean concrete (m)
Retaining wall canal (Masonry)	0.063	1/1000 =0.001	0.015	0.50	0.24	1:0	0.52	0.181	0.424	0.45	0.1	0.1
Earthen canal (Clay soil)	0.063	2/1000 =0.002	0.025	0.15	0.30	1:1	0.47	0.176	0.476	0.50	-	-

<In case of supercritical or rapid flow canal >

Slope = 7/100 = 0.07

Minimum allowable velocity = 0.45m/s

Maximum allowable velocity = 2.25m/s

(Velocity of Thick concrete (approximately 14 cm) = (Velocity of Thick concrete (approximately 18 cm) + Velocity of Thick concrete (approximately 10 cm)) / 2 = (3 + 1.5) / 2 = 2.25m/s)

Type of canal	Discharge (m3/s)	Slope	Coefficient of Roughness	Canal width (m)	Water depth (m)	Side slope	Velocity (m/s)	Free board (m)	Necessary canal height (m)	Design canal height (m)	Hard coring (m)	Lean concrete (m)
Rapid flow canal (concrete lining)	0.063	7/100 =0.07	0.015 Concrete (cast-in place flume, culvert, etc.))	0.50	0.052	1:1	2.21	0.065 - 0.63	0.117 - 0.682	0.30	0.1	-

This result should be checked and modified according to the actual site condition. Especially it should be checked the possibility of excavation until the canal bed based on geological data.

(5) Stability analysis of retaining wall canal...... (See 3.3.2, 3.3.3, 3.3.4, 3.3.5)

For stability analysis of retaining wall canal can be used Excel format guide 2. There are three types of calculation for stability analysis according to the ground condition of back side of canal.

The result of calculation, the size of retaining wall canal does not have any problem.

Excel format guide 2. Wall Stability Analysis Type1 (Summary)

#Please input data into yellow cell

1.Basic data for calculation

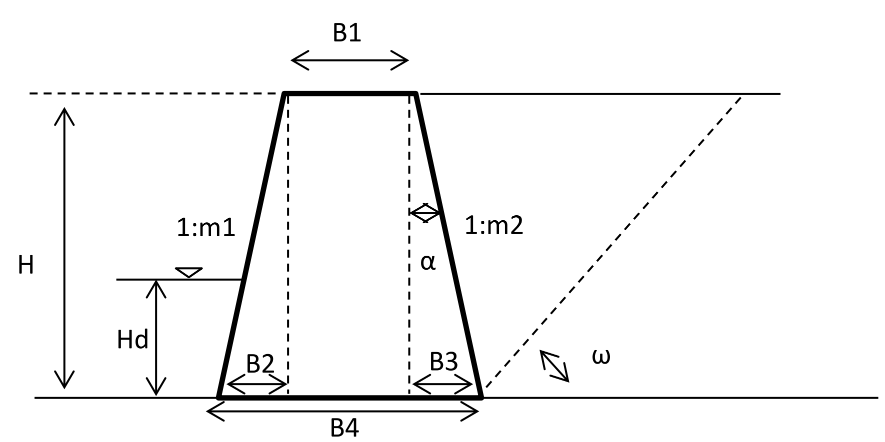
Item		Value			Reference
Unit weight of plane concrete	γc=	23	kN/m^3		Reinforced concrete: 24.5KN/m3, Plain concrete: 23KN/m3, Cement mortar: 21KN/m3
Unit weight of soil	γs=	20	kN/m^3		
internal friction angle	φ=	25	°	=0.436rad	
Angle between wall's back surface and vertical plane	α=	0.0	°	=0.000rad	
Angle of wall friction (normal condition)	δ=	16.7	°	=0.291rad	
Angle of wall friction (seismic condition)	δ=	12.5	°	=0.218rad	
Liveload	q=	3.00	kN/m^2		
Unit weight of water	W_0=	9.8	kN/m^3		
Coefficient of earth pressure	C_0=	0.45			
Uplift coefficient	μ=	1.0			Rock foundation case or a case using sheet piles reaching an impermeable stratum: 0.4, Otherwise: 1.0
Seismic horizontal acceleration	kh=	0.15			
Seismic vertical acceleration	kv=	0			
Seismic compound angle	θ=	8.53	°	=0.149rad	
Friction coefficient	f=	0.7			
Allowable stress of the graund	qa=	50	kN/m^2		Bedrock=1000KN/m2 See 3.2.4

2.Mesurement of structure

Item		Value	
Slope of front body	m1 = 1 :	0.00	
Slope of back body	m2 = 1 :	0.00	
B1		0.30	m
B2		0.00	m
B3		0.00	m
B4 (B1+B2+B3)		0.30	m
H		0.45	m
H_d		0.45	m

3. The result of calculation

Content		Dynamic				Static				Earthquake			
Overturning	e<B/6 or B/3	0.02	≤	0.05	OK	0.04	≤	0.05	OK	0.08	≤	0.10	OK
Sliding	ΣV･f/ΣH≥1.5or1.2	10.92	≥	1.50	OK	2.14	≥	1.50	OK	1.20	≥	1.20	OK
Settlement	ΣV/B･(1+6e/B)<q_a	13kN/m2	<	50kN/m2	OK	20kN/m2	<	50kN/m2	OK	32kN/m2	<	50kN/m2	OK
	ΣV/B･(1-6e/B)<q_a	6kN/m2	<	50kN/m2	OK	4kN/m2	<	50kN/m2	OK	-8kN/m2	<	50kN/m2	OK
(Without uplift)	ΣV/B･(1+6e/B)<q_a	14kN/m2	<	50kN/m2	OK								
(Without uplift)	ΣV/B･(1-6e/B)<q_a	10kN/m2	<	50kN/m2	OK								



Type 1

①Calculation of earth pressure (The case of dynamic)

1. Calculation of active earth pressure

$$\frac{P_A}{\sin(\omega-\phi)}=\frac{W}{\sin\{90^\circ-(\omega-\phi-\delta-\alpha)}$$

$$\therefore P_A=\frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)}W \qquad (7.1.14)$$

where P_A: Active earth pressure (kN/m)
W: Weight of soil wedge (including overburden load) (kN/m)
ϕ: Angle of shear resistance of backside ground (°)
α: Angle between wall's back surface and vertical plane (°) (Figure 7.1.10)
δ: Angle of wall friction (°) (Table 7.1.4)
ω: Angle between the assumed sliding surface and horizontal plane (°)

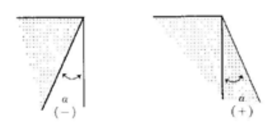
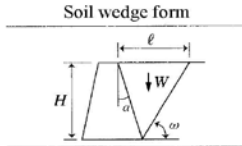
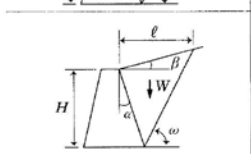
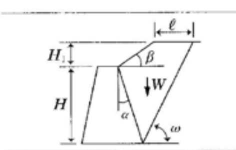
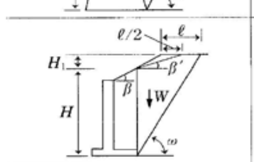
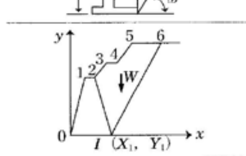


Figure 7.1.10 Illustrated examples of positive and negative values for α

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot H$
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)\cdot\cos(\alpha-\beta)}{\cos^2\alpha\cdot\sin(\omega-\beta)}$ $\ell=\frac{\cos\beta\cdot\cos(\omega-\alpha)}{\cos\alpha\cdot\sin(\omega-\beta)}\cdot H$
	$W=\frac{1}{2}\gamma\left\{(H+H_1)^2\frac{\cos(\omega-\alpha)}{\sin\omega}-H_1{}^2\frac{\cos(\alpha-\beta)}{\sin\beta}\right\}\frac{1}{\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot(H+H_1)-\frac{\cos(\beta-\alpha)}{\sin\beta\cdot\cos\alpha}\cdot H_1$
	$W=\frac{1}{2}\gamma\ \{(H+H_1)^2\cot\omega-H_1{}^2\cot\beta\}$ $\beta'=\tan^{-1}\frac{2H_1}{(H+H_1)\cot\omega+H_1\cot\beta}$ $\ell=(H+H_1)\cot\omega-H_1\cot\beta$
	$W=\frac{1}{2}\gamma\sum_{i=1}^{n}(X_{i+1}-X_i)(Y_{i+1}+Y_1)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	4.342627	0.965028	2.895084	7.237711	0	56
26.00	0.454	4.151865	0.922637	2.76791	6.919775	0.125426	55
27.00	0.471	3.974286	0.883175	2.649524	6.62381	0.238954	54
28.00	0.489	3.808471	0.846327	2.538981	6.347452	0.34188	53
29.00	0.506	3.653197	0.811821	2.435464	6.088661	0.435318	52
30.00	0.524	3.507403	0.779423	2.338269	5.845671	0.520231	51
31.00	0.541	3.370166	0.748926	2.246777	5.616943	0.597454	50
32.00	0.559	3.240677	0.720151	2.160452	5.401129	0.667713	47
33.00	0.576	3.118227	0.692939	2.078818	5.197044	0.731643	45
34.00	0.593	3.002186	0.667152	2.001457	5.003643	0.789802	43
35.00	0.611	2.892	0.642667	1.928	4.82	0.842682	40
36.00	0.628	2.787173	0.619372	1.858116	4.645289	0.890716	38
37.00	0.646	2.687266	0.59717	1.791511	4.478776	0.934287	35
38.00	0.663	2.591882	0.575974	1.727921	4.319803	0.973737	33
39.00	0.681	2.500667	0.555704	1.667111	4.167778	1.00937	31
40.00	0.698	2.413301	0.536289	1.608867	4.022168	1.041454	28
41.00	0.716	2.329496	0.517666	1.552997	3.882493	1.070233	26
42.00	0.733	2.24899	0.499776	1.499327	3.748317	1.09592	24
43.00	0.750	2.171547	0.482566	1.447698	3.619244	1.118711	22
44.00	0.768	2.096949	0.465989	1.397966	3.494915	1.138777	19
45.00	0.785	2.025	0.45	1.35	3.375	1.156274	17
46.00	0.803	1.95552	0.43456	1.30368	3.2592	1.171341	15
47.00	0.820	1.888343	0.419632	1.258895	3.147238	1.184102	13
48.00	0.838	1.823318	0.405182	1.215545	3.038864	1.19467	11
49.00	0.855	1.760306	0.391179	1.173537	2.933843	1.203143	9
50.00	0.873	1.699177	0.377595	1.132785	2.831961	1.20961	7
51.00	0.890	1.639813	0.364403	1.093208	2.733021	1.214151	5
52.00	0.908	1.582103	0.351579	1.054736	2.636839	1.216836	3
53.00	0.925	1.525947	0.339099	1.017298	2.543245	1.217726	1
54.00	0.942	1.471249	0.326944	0.980832	2.452081	1.216876	2
55.00	0.960	1.41792	0.315093	0.94528	2.3632	1.214333	4
56.00	0.977	1.36588	0.303529	0.910586	2.276466	1.210136	6
57.00	0.995	1.31505	0.292233	0.8767	2.191751	1.20432	8
58.00	1.012	1.26536	0.281191	0.843574	2.108934	1.196913	10
59.00	1.030	1.216743	0.270387	0.811162	2.027905	1.187937	12

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
60.00	1.047198	1.169134	0.259808	0.779423	1.948557	1.177409	14
61.00	1.064651	1.122476	0.249439	0.748317	1.870793	1.16534	16
62.00	1.082104	1.076712	0.239269	0.717808	1.794519	1.151737	18
63.00	1.099557	1.031789	0.229286	0.687859	1.719648	1.136601	20
64.00	1.117011	0.987658	0.21948	0.658439	1.646097	1.11993	21
65.00	1.134464	0.944273	0.209838	0.629515	1.573788	1.101714	23
66.00	1.151917	0.901588	0.200353	0.601059	1.502647	1.081941	25
67.00	1.169371	0.859562	0.191014	0.573041	1.432603	1.060591	27
68.00	1.186824	0.818153	0.181812	0.545435	1.363589	1.037643	29
69.00	1.204277	0.777325	0.172739	0.518216	1.295541	1.013067	30
70.00	1.22173	0.73704	0.163787	0.49136	1.2284	0.986831	32
71.00	1.239184	0.697263	0.154947	0.464842	1.162106	0.958894	34
72.00	1.256637	0.657962	0.146214	0.438642	1.096604	0.929213	36
73.00	1.27409	0.619105	0.137579	0.412736	1.031841	0.897736	37
74.00	1.291544	0.580659	0.129035	0.387106	0.967766	0.864408	39
75.00	1.308997	0.542597	0.120577	0.361731	0.904329	0.829163	41
76.00	1.32645	0.504889	0.112198	0.336593	0.841482	0.791933	42
77.00	1.343904	0.467508	0.103891	0.311672	0.77918	0.752637	44
78.00	1.361357	0.430427	0.09565	0.286951	0.717378	0.711191	46
79.00	1.37881	0.39362	0.087471	0.262413	0.656034	0.667499	48
80.00	1.396263	0.357062	0.079347	0.238041	0.595104	0.621457	49

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	53 °
Active earth pressure (P_A)	1.22 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{array}{l} P_{AH} = P_A \cos(\delta + \alpha) \\ P_{AV} = P_A \sin(\delta + \alpha) \end{array}\right\} \quad \text{......(7.1.15)}$$

$$h_p = \frac{1}{3} H \quad \text{......(7.1.16)}$$

where P_{AH}: Horizontal component of active earth pressure (kN/m)
P_{AV}: Vertical component of active earth pressure (kN/m)
h_p: Vertical distance to the action point of active earth pressure (m)
H: Height of wall (m)

P_{AH}=	1.17 KN/m
P_{AV}=	0.35 KN/m
h_p=	0.15 m

②Stability analysis (The case of dynamic)

1.Basic calculation

(1)Area

External force		Area				
Dead load;	W1	0.14m2=	0.30m×	0.45m		
	W2	0.00m2=	0.00m×	0.45m×	1/2	
	W3	0.00m2=	0.00m×	0.45m×	1/2	
Static water pressure;	P1	0.10m2=	0.45m×	0.45m×	1/2	
Uplift;	U1	0.07m2=	0.45m×	0.30m×	1/2	
Earth pressure	P_{AH}					
	P_{AV}					

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;	P1	0.15m=	0.45m×	1/3		
Uplift;	U1	0.10m=	0.30m×	1/3		
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)

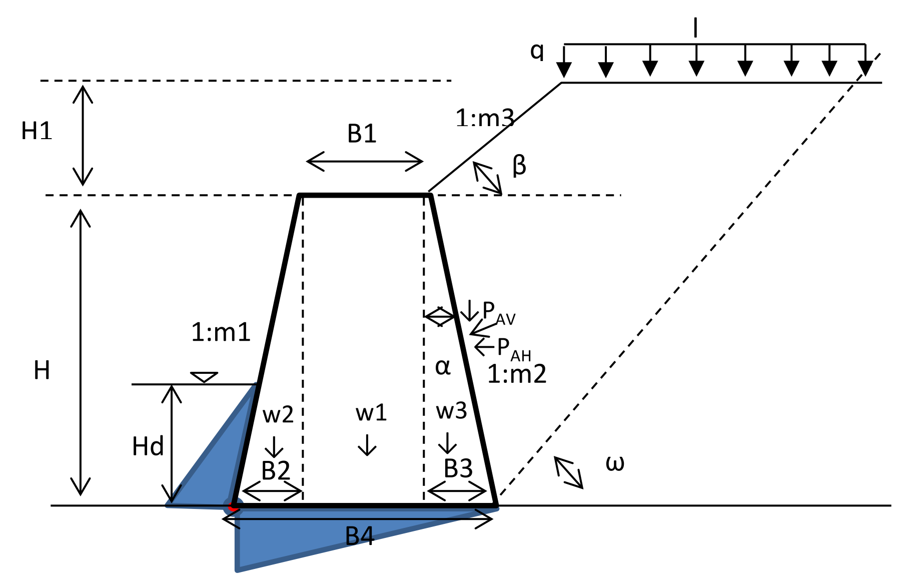
(3)External force by unit width

External force		External force by unit width			
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2	
	W2	0.00kN=	0.00m2×	23.00kN/m2	
	W3	0.00kN=	0.00m2×	23.00kN/m2	
Static water pressure;	P1	1.0kN=	0.10m2×	9.80kN/m2	
Uplift;	U1	-0.66kN=	0.07m2×	-9.80kN/m2	×1.0
Earth pressure;	P_{AH}	-1.17kN			
	P_{AV}	0.35kN			

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure	P1				0.98	0.15	0.15
Uplift	U1	-0.66	0.10	-0.07			
Earth pressure	P_{AH}				-1.17	0.15	-0.17
	P_{AV}	0.35	0.30	0.10			
Total		2.91		0.51	-0.19		-0.02



Type 1

①Calculation of earth pressure (The case of static)

1. Calculation of active earth pressure

$$\frac{P_A}{\sin(\omega-\phi)} = \frac{W}{\sin\{90° - (\omega-\phi-\delta-\alpha)}$$

$$\therefore P_A = \frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)} W \quad \text{.......(7.1.14)}$$

where P_A: Active earth pressure (kN/m)
W: Weight of soil wedge (including overburden load) (kN/m)
ϕ: Angle of shear resistance of backside ground (°)
α: Angle between wall's back surface and vertical plane (°) (Figure 7.1.10)
δ: Angle of wall friction (°) (Table 7.1.4)
ω: Angle between the assumed sliding surface and horizontal plane (°)

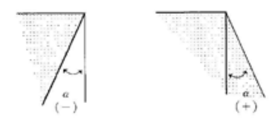
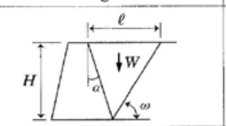
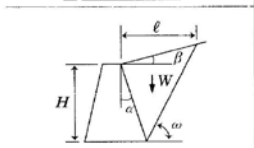
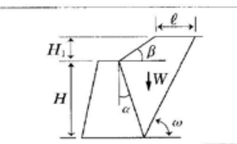
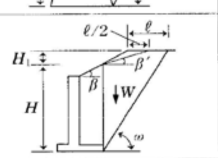
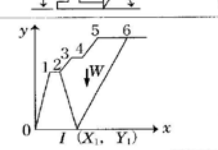


Figure 7.1.10 Illustrated examples of positive and negative values for α

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot H$
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha) \cdot \cos(\alpha-\beta)}{\cos^2\alpha \cdot \sin(\omega-\beta)}$ $\ell = \frac{\cos\beta \cdot \cos(\omega-\alpha)}{\cos\alpha \cdot \sin(\omega-\beta)} \cdot H$
	$W = \frac{1}{2}\gamma \left\{ (H+H_1)^2 \frac{\cos(\omega-\alpha)}{\sin\omega} - H_1{}^2 \frac{\cos(\alpha-\beta)}{\sin\beta} \right\} \frac{1}{\cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot (H+H_1) - \frac{\cos(\beta-\alpha)}{\sin\beta \cdot \cos\alpha} \cdot H_1$
	$W = \frac{1}{2}\gamma \{(H+H_1)^2 \cot\omega - H_1{}^2 \cot\beta\}$ $\beta' = \tan^{-1} \frac{2H_1}{(H+H_1)\cot\omega + H_1\cot\beta}$ $\ell = (H+H_1)\cot\omega - H_1\cot\beta$
	$W = \frac{1}{2}\gamma \sum_{i=1}^{n} (X_{i+1} - X_i)(Y_{i+1} + Y_i)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	4.342627	0.965028	2.895084	7.237711	0	56
26.00	0.454	4.151865	0.922637	2.76791	6.919775	0.125426	55
27.00	0.471	3.974286	0.883175	2.649524	6.62381	0.238954	54
28.00	0.489	3.808471	0.846327	2.538981	6.347452	0.34188	53
29.00	0.506	3.653197	0.811821	2.435464	6.088661	0.435318	52
30.00	0.524	3.507403	0.779423	2.338269	5.845671	0.520231	51
31.00	0.541	3.370166	0.748926	2.246777	5.616943	0.597454	50
32.00	0.559	3.240677	0.720151	2.160452	5.401129	0.667713	47
33.00	0.576	3.118227	0.692939	2.078818	5.197044	0.731643	45
34.00	0.593	3.002186	0.667152	2.001457	5.003643	0.789802	43
35.00	0.611	2.892	0.642667	1.928	4.82	0.842682	40
36.00	0.628	2.787173	0.619372	1.858116	4.645289	0.890716	38
37.00	0.646	2.687266	0.59717	1.791511	4.478776	0.934287	35
38.00	0.663	2.591882	0.575974	1.727921	4.319803	0.973737	33
39.00	0.681	2.500667	0.555704	1.667111	4.167778	1.00937	31
40.00	0.698	2.413301	0.536289	1.608867	4.022168	1.041454	28
41.00	0.716	2.329496	0.517666	1.552997	3.882493	1.070233	26
42.00	0.733	2.24899	0.499776	1.499327	3.748317	1.09592	24
43.00	0.750	2.171547	0.482566	1.447698	3.619244	1.118711	22
44.00	0.768	2.096949	0.465989	1.397966	3.494915	1.138777	19
45.00	0.785	2.025	0.45	1.35	3.375	1.156274	17
46.00	0.803	1.95552	0.43456	1.30368	3.2592	1.171341	15
47.00	0.820	1.888343	0.419632	1.258895	3.147238	1.184102	13
48.00	0.838	1.823318	0.405182	1.215545	3.038864	1.19467	11
49.00	0.855	1.760306	0.391179	1.173537	2.933843	1.203143	9
50.00	0.873	1.699177	0.377595	1.132785	2.831961	1.20961	7
51.00	0.890	1.639813	0.364403	1.093208	2.733021	1.214151	5
52.00	0.908	1.582103	0.351579	1.054736	2.636839	1.216836	3
53.00	0.925	1.525947	0.339099	1.017298	2.543245	1.217726	1
54.00	0.942	1.471249	0.326944	0.980832	2.452081	1.216876	2
55.00	0.960	1.41792	0.315093	0.94528	2.3632	1.214333	4
56.00	0.977	1.36588	0.303529	0.910586	2.276466	1.210136	6
57.00	0.995	1.31505	0.292233	0.8767	2.191751	1.20432	8
58.00	1.012	1.26536	0.281191	0.843574	2.108934	1.196913	10
59.00	1.030	1.216743	0.270387	0.811162	2.027905	1.187937	12

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
60.00	1.047198	1.169134	0.259808	0.779423	1.948557	1.177409	14
61.00	1.064651	1.122476	0.249439	0.748317	1.870793	1.16534	16
62.00	1.082104	1.076712	0.239269	0.717808	1.794519	1.151737	18
63.00	1.099557	1.031789	0.229286	0.687859	1.719648	1.136601	20
64.00	1.117011	0.987658	0.21948	0.658439	1.646097	1.11993	21
65.00	1.134464	0.944273	0.209838	0.629515	1.573788	1.101714	23
66.00	1.151917	0.901588	0.200353	0.601059	1.502647	1.081941	25
67.00	1.169371	0.859562	0.191014	0.573041	1.432603	1.060591	27
68.00	1.186824	0.818153	0.181812	0.545435	1.363589	1.037643	29
69.00	1.204277	0.777325	0.172739	0.518216	1.295541	1.013067	30
70.00	1.22173	0.73704	0.163787	0.49136	1.2284	0.986831	32
71.00	1.239184	0.697263	0.154947	0.464842	1.162106	0.958894	34
72.00	1.256637	0.657962	0.146214	0.438642	1.096604	0.929213	36
73.00	1.27409	0.619105	0.137579	0.412736	1.031841	0.897736	37
74.00	1.291544	0.580659	0.129035	0.387106	0.967766	0.864408	39
75.00	1.308997	0.542597	0.120577	0.361731	0.904329	0.829163	41
76.00	1.32645	0.504889	0.112198	0.336593	0.841482	0.791933	42
77.00	1.343904	0.467508	0.103891	0.311672	0.77918	0.752637	44
78.00	1.361357	0.430427	0.09565	0.286951	0.717378	0.711191	46
79.00	1.37881	0.39362	0.087471	0.262413	0.656034	0.667499	48
80.00	1.396263	0.357062	0.079347	0.238041	0.595104	0.621457	49

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	53 °
Active earth pressure (P_A)	1.22 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{aligned} P_{AH} &= P_A \cos(\delta + \alpha) \\ P_{AV} &= P_A \sin(\delta + \alpha) \end{aligned}\right\} \quad \cdots\cdots (7.1.15)$$

$$h_p = \frac{1}{3}H \quad \cdots\cdots (7.1.16)$$

where
P_{AH}: Horizontal component of active earth pressure (kN/m)
P_{AV}: Vertical component of active earth pressure (kN/m)
h_p: Vertical distance to the action point of active earth pressure (m)
H: Height of wall (m)

P_{AH}=	1.17 KN/m
P_{AV}=	0.35 KN/m
h_p=	0.15 m

②Stability analysis (The case of static)

1.Basic calculation

(1)Area

External force		Area				
Dead load;	W1	0.14m2=	0.30m×	0.45m		
	W2	0.00m2=	0.00m×	0.45m×	1/2	
	W3	0.00m2=	0.00m×	0.45m×	1/2	
Static water pressure;						
Uplift;						
Earth pressure	P_{AH}					
	P_{AV}					

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;						
Uplift;						
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)

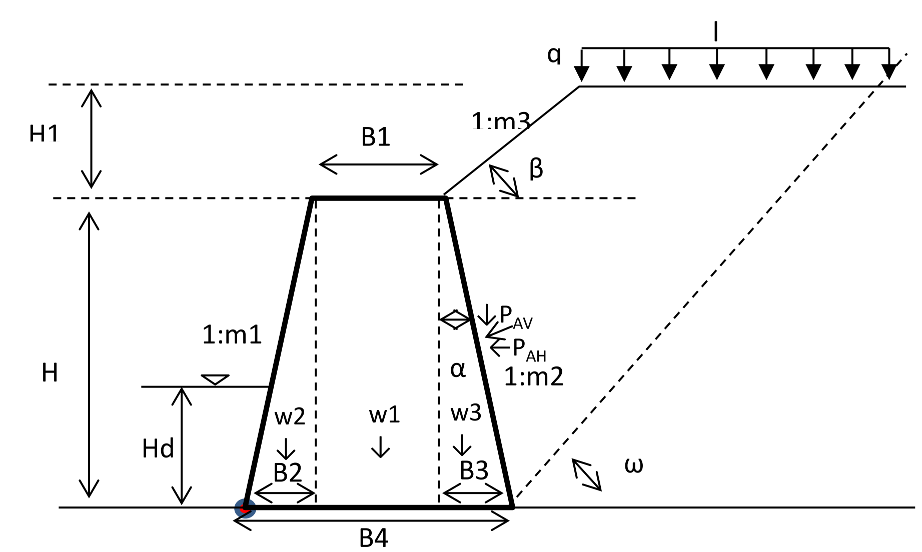
(3)External force by unit width

External force		External force by unit width		
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2
	W2	0.00kN=	0.00m2×	23.00kN/m2
	W3	0.00kN=	0.00m2×	23.00kN/m2
Static water pressure;				
Uplift;				
Earth pressure;	P_{AH}	-1.17kN		
	P_{AV}	0.35kN		

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure							
Uplift							
Earth pressure	P_{AH}				-1.17	0.15	-0.17
	P_{AV}	0.35	0.30	0.10			
Total		3.57		0.58	-1.17		-0.17



Type 1

①Calculation of earth pressure (The case of earthquick)

1. Calculation of active earth pressure

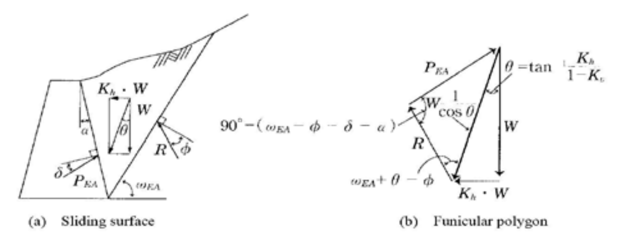


Figure 7.1.15 Concept of active earth pressure under seismic condition

$$P_{EA} = \frac{\sin(\omega_{EA} - \phi + \theta)}{\cos(\omega_{EA} - \phi - \delta - \alpha)\cdot\cos\theta} W \quad \text{.......(7.1.18)}$$

$$\left.\begin{array}{l} P_{EAH} = P_{EA}\cos(\delta + \alpha) \\ P_{EAV} = P_{EA}\sin(\delta + \alpha) \end{array}\right\} \quad \text{.......(7.1.19)}$$

$$h_p = \frac{1}{3}H \quad \text{.......(7.1.20)}$$

where P_{EA}: Active earth pressure under seismic condition (kN/m)

P_{EAH}: Horizontal component of active earth pressure under seismic condition (kN/m)

P_{EAV}: Vertical component of active earth pressure under seismic condition (kN/m)

ω_{EA}: Angle between assumed sliding surface and horizontal plane (°)

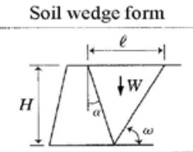
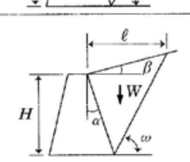
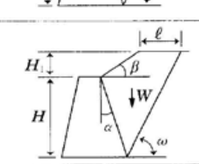
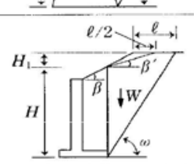
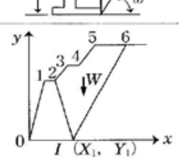
θ: Seismic compound angle ($\tan^{-1}\frac{K_h}{1-K_v}$) (°)

K_h: Design horizontal seismic coefficient (see the equation (7.1.44))

δ: Angle of wall friction under seismic condition (°) (see the equation (7.1.4))

h_p: Vertical distance to the action point of active earth pressure under seismic condition (m)

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot H$
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha) \cdot \cos(\alpha-\beta)}{\cos^2\alpha \cdot \sin(\omega-\beta)}$ $\ell = \frac{\cos\beta \cdot \cos(\omega-\alpha)}{\cos\alpha \cdot \sin(\omega-\beta)} \cdot H$
	$W = \frac{1}{2}\gamma \left\{ (H+H_1)^2 \frac{\cos(\omega-\alpha)}{\sin\omega} - H_1{}^2 \frac{\cos(\alpha-\beta)}{\sin\beta} \right\} \frac{1}{\cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot (H+H_1) - \frac{\cos(\beta-\alpha)}{\sin\beta \cdot \cos\alpha} \cdot H_1$
	$W = \frac{1}{2}\gamma \{(H+H_1)^2\cot\omega - H_1{}^2\cot\beta\}$ $\beta' = \tan^{-1}\frac{2H_1}{(H+H_1)\cot\omega + H_1\cot\beta}$ $\ell = (H+H_1)\cot\omega - H_1\cot\beta$
	$W = \frac{1}{2}\gamma \sum_{i=1}^{n} (X_{i+1} - X_i)(Y_{i+1} + Y_1)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	4.342627	0.965028	2.895084	7.237711	1.112016	47
26.00	0.454	4.151865	0.922637	2.76791	6.919775	1.18231	45
27.00	0.471	3.974286	0.883175	2.649524	6.62381	1.244981	42
28.00	0.489	3.808471	0.846327	2.538981	6.347452	1.300853	39
29.00	0.506	3.653197	0.811821	2.435464	6.088661	1.350634	36
30.00	0.524	3.507403	0.779423	2.338269	5.845671	1.394932	34
31.00	0.541	3.370166	0.748926	2.246777	5.616943	1.434276	31
32.00	0.559	3.240677	0.720151	2.160452	5.401129	1.469126	28
33.00	0.576	3.118227	0.692939	2.078818	5.197044	1.499882	26
34.00	0.593	3.002186	0.667152	2.001457	5.003643	1.526896	24
35.00	0.611	2.892	0.642667	1.928	4.82	1.550476	21
36.00	0.628	2.787173	0.619372	1.858116	4.645289	1.570893	19
37.00	0.646	2.687266	0.59717	1.791511	4.478776	1.588386	17
38.00	0.663	2.591882	0.575974	1.727921	4.319803	1.603168	14
39.00	0.681	2.500667	0.555704	1.667111	4.167778	1.615427	12
40.00	0.698	2.413301	0.536289	1.608867	4.022168	1.625328	10
41.00	0.716	2.329496	0.517666	1.552997	3.882493	1.63302	8
42.00	0.733	2.24899	0.499776	1.499327	3.748317	1.638633	6
43.00	0.750	2.171547	0.482566	1.447698	3.619244	1.642285	4
44.00	0.768	2.096949	0.465989	1.397966	3.494915	1.644077	2
45.00	0.785	2.025	0.45	1.35	3.375	1.644103	1
46.00	0.803	1.95552	0.43456	1.30368	3.2592	1.642442	3
47.00	0.820	1.888343	0.419632	1.258895	3.147238	1.639167	5
48.00	0.838	1.823318	0.405182	1.215545	3.038864	1.634339	7
49.00	0.855	1.760306	0.391179	1.173537	2.933843	1.628014	9
50.00	0.873	1.699177	0.377595	1.132785	2.831961	1.620239	11
51.00	0.890	1.639813	0.364403	1.093208	2.733021	1.611055	13
52.00	0.908	1.582103	0.351579	1.054736	2.636839	1.600496	15
53.00	0.925	1.525947	0.339099	1.017298	2.543245	1.58859	16
54.00	0.942	1.471249	0.326944	0.980832	2.452081	1.575362	18
55.00	0.960	1.41792	0.315093	0.94528	2.3632	1.560829	20
56.00	0.977	1.36588	0.303529	0.910586	2.276466	1.545004	22
57.00	0.995	1.31505	0.292233	0.8767	2.191751	1.527895	23
58.00	1.012	1.26536	0.281191	0.843574	2.108934	1.509507	25
59.00	1.030	1.216743	0.270387	0.811162	2.027905	1.489838	27
60.00	1.047	1.169134	0.259808	0.779423	1.948557	1.468883	29
61.00	1.065	1.122476	0.249439	0.748317	1.870793	1.446634	30
62.00	1.082	1.076712	0.239269	0.717808	1.794519	1.423076	32
63.00	1.100	1.031789	0.229286	0.687859	1.719648	1.398191	33
64.00	1.117	0.987658	0.21948	0.658439	1.646097	1.371957	35

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
65.00	1.134464	0.944273	0.209838	0.629515	1.573788	1.344347	37
66.00	1.151917	0.901588	0.200353	0.601059	1.502647	1.31533	38
67.00	1.169371	0.859562	0.191014	0.573041	1.432603	1.284869	40
68.00	1.186824	0.818153	0.181812	0.545435	1.363589	1.252923	41
69.00	1.204277	0.777325	0.172739	0.518216	1.295541	1.219446	43
70.00	1.22173	0.73704	0.163787	0.49136	1.2284	1.184386	44
71.00	1.239184	0.697263	0.154947	0.464842	1.162106	1.147686	46
72.00	1.256637	0.657962	0.146214	0.438642	1.096604	1.109281	48
73.00	1.27409	0.619105	0.137579	0.412736	1.031841	1.069102	49
74.00	1.291544	0.580659	0.129035	0.387106	0.967766	1.027072	50
75.00	1.308997	0.542597	0.120577	0.361731	0.904329	0.983105	51
76.00	1.32645	0.504889	0.112198	0.336593	0.841482	0.937108	52
77.00	1.343904	0.467508	0.103891	0.311672	0.77918	0.88898	53
78.00	1.361357	0.430427	0.09565	0.286951	0.717378	0.838609	54
79.00	1.37881	0.39362	0.087471	0.262413	0.656034	0.785872	55
80.00	1.396263	0.357062	0.079347	0.238041	0.595104	0.730635	56

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	45 °
Active earth pressure (P_A)	1.64 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{aligned} P_{AH} &= P_A \cos(\delta + \alpha) \\ P_{AV} &= P_A \sin(\delta + \alpha) \end{aligned}\right\} \quad \text{......(7.1.15)}$$

$$h_p = \frac{1}{3} H \quad \text{......(7.1.16)}$$

where P_{AH}: Horizontal component of active earth pressure (kN/m)
P_{AV}: Vertical component of active earth pressure (kN/m)
h_p: Vertical distance to the action point of active earth pressure (m)
H: Height of wall (m)

P_{AH}=	1.61 KN/m
P_{AV}=	0.36 KN/m
h_p=	0.15 m

②Stability analysis (The case of earthquick)

1.Basic calculation

(1)Area

External force		Area			
Dead load;	W1	0.14m2=	0.30m×	0.45m	
	W2	0.00m2=	0.00m×	0.45m×	1/2
	W3	0.00m2=	0.00m×	0.45m×	1/2
Static water pressure;					
Uplift;					
Earth pressure	P_{AH}				
	P_{AV}				

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;						
Uplift;						
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)
	S1	0.23m=	0.45m×	1/2		
	S2	0.15m=	0.45m×	1/3		
	S3	0.15m=	0.45m×	1/3		

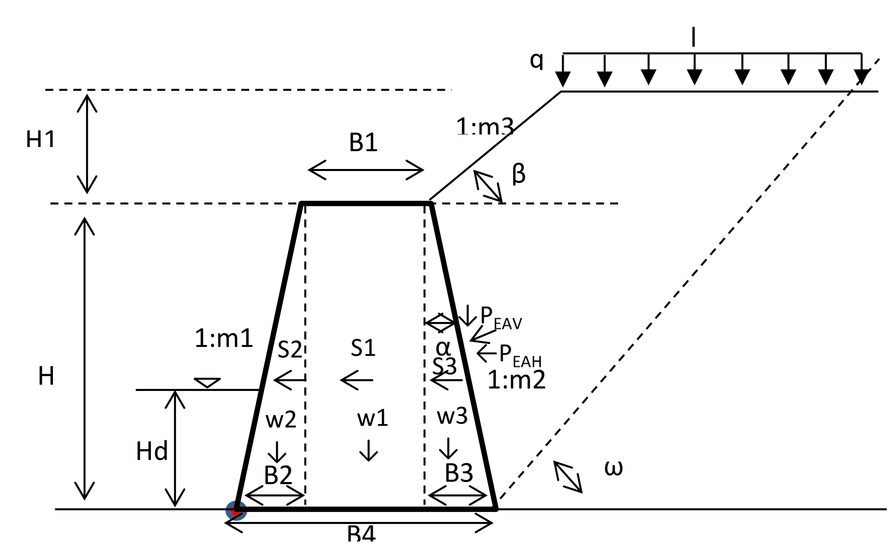
(3)External force by unit width

External force		External force by unit width		
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2
	W2	0.00kN=	0.00m2×	23.00kN/m2
	W3	0.00kN=	0.00m2×	23.00kN/m2
Static water pressure;				
Uplift;				
Earth pressure;	P_{EAH}	-1.61kN		
	P_{EAV}	0.36kN		
	S1	-0.48kN=	-3.22KN×	0.15
	S2	0.00kN=	0.00KN×	0.15
	S3	0.00kN=	0.00KN×	0.15

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure	P1						
Uplift	U1						
Earth pressure	P_{AH}				-1.61	0.15	-0.24
	P_{AV}	0.36	0.30	0.11			
	S1				-0.48	0.23	-0.11
	S2				0.00	0.15	0.00
	S3				0.00	0.15	0.00
Total		3.58		0.59	-2.09		-0.35



Excel format guide 2. Wall Stability Analysis Type2 (Summary)

#Please input data into yellow cell

1.Basic data for calculation

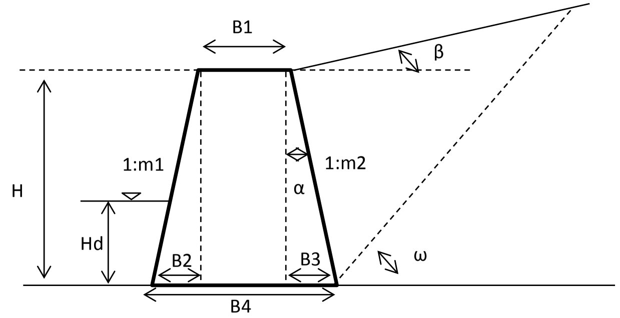
Item	Value				Reference
Unit weight of plane concrete	γc=	23	kN/m^3		Reinforced concrete: 24.5KN/m3, Plain concrete: 23KN/m3, Cement mortar: 21KN/m3
Unit weight of soil	γs=	20	kN/m^3		
internal friction angle	ϕ=	25	°	=0.436rad	
Angle between wall's back surface and vertical plane	α=	0.0	°	=0.000rad	
Inclination angle of wall's backside ground	β=	15	°	=0.262rad	
Angle of wall friction (normal condition)	δ=	16.7	°	=0.291rad	
Angle of wall friction (seismic condition)	δ=	12.5	°	=0.218rad	
Crowd load	q=	3.00	kN/m^2		
Unit weight of water	W_0=	9.8	kN/m^3		
Coefficient of earth pressure	C_0=	0.45			
Uplift coefficient	μ=	1.0			Rock foundation case or a case using sheet piles reaching an impermeable stratum: 0.4, Otherwise: 1.0
Seismic horizontal acceleration	kh=	0			
Seismic vertical acceleration	kv=	0			
Seismic compound angle	θ=	0.00	°	=0.000rad	
Friction coefficient	f=	0.7			
Allowable stress of the graund	qa=	50	kN/m^2		Bedrock=1000KN/m2 See 3.2.4

2.Mesurement of structure

Item		Value	
Slope of front body	m1 = 1 :	0.00	
Slope of back body	m2 = 1 :	0.00	
B1		0.30	m
B2		0.00	m
B3		0.00	m
B4 (B1+B2+B3)		0.30	m
H		0.45	m
H_d		0.45	m

3. The result of calculation

Content		Dynamic				Static				Earthquake			
Overturning	e<B/6 or B/3	0.01	≤	0.05	OK	0.04	≤	0.05	OK	0.05	≤	0.10	OK
Sliding	ΣV･f/ΣH≥1.5or1.2	5.65	≥	1.50	OK	1.88	≥	1.50	OK	1.78	≥	1.20	OK
Settlement	ΣV/B･(1+6e/B)<q_a	12kN/m2	<	50kN/m2	OK	22kN/m2	<	50kN/m2	OK	23kN/m2	<	50kN/m2	OK
	ΣV/B･(1-6e/B)<q_a	8kN/m2	<	50kN/m2	OK	3kN/m2	<	50kN/m2	OK	0kN/m2	<	50kN/m2	OK
(Without uplift)	ΣV/B･(1+6e/B)<q_a	13kN/m2	<	50kN/m2	OK								
(Without uplift)	ΣV/B･(1-6e/B)<q_a	12kN/m2	<	50kN/m2	OK								



Type 2

①Calculation of earth pressure (The case of dynamic)

1. Calculation of active earth pressure

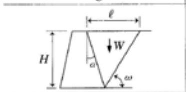
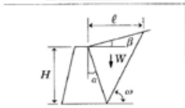
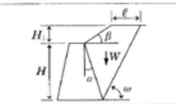
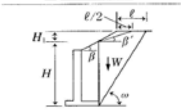
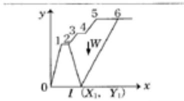
$$\frac{P_A}{\sin(\omega-\phi)}=\frac{W}{\sin\{90°-(\omega-\phi-\delta-\alpha)\}}$$

$$\therefore P_A=\frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)}W \quad \text{(7.1.14)}$$

where P_A: Active earth pressure (kN/m)
W: Weight of soil wedge (including overburden load) (kN/m)
ϕ: Angle of shear resistance of backside ground (°)
α: Angle between wall's back surface and vertical plane (°) (Figure 7.1.10)
δ: Angle of wall friction (°) (Table 7.1.4)
ω: Angle between the assumed sliding surface and horizontal plane (°)

Figure 7.1.10 Illustrated examples of positive and negative values for α

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot H$
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)\cdot\cos(\alpha-\beta)}{\cos^2\alpha\cdot\sin(\omega-\beta)}$ $\ell=\frac{\cos\beta\cdot\cos(\omega-\alpha)}{\cos\alpha\cdot\sin(\omega-\beta)}\cdot H$
	$W=\frac{1}{2}\gamma\left\{(H+H_1)^2\frac{\cos(\omega-\alpha)}{\sin\omega}-H_1^2\frac{\cos(\alpha-\beta)}{\sin\beta}\right\}\frac{1}{\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot(H+H_1)-\frac{\cos(\beta-\alpha)}{\sin\beta\cdot\cos\alpha}\cdot H_1$
	$W=\frac{1}{2}\gamma\{(H+H_1)^2\cot\omega-H_1^2\cot\beta\}$ $\beta'=\tan^{-1}\frac{2H_1}{(H+H_1)\cot\omega+H_1\cot\beta}$ $\ell=(H+H_1)\cot\omega-H_1\cot\beta$
	$W=\frac{1}{2}\gamma\sum_{i=1}^{n}(X_{i+1}-X_i)(Y_{i+1}+Y_i)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	10.20879	2.26862	6.80586	17.01465	0	56
26.00	0.454	9.213617	2.408933	7.226799	16.44042	0.297996	55
27.00	0.471	8.382446	1.546812	4.640436	13.02288	0.4698	54
28.00	0.489	7.677427	0.817489	2.452467	10.12989	0.545606	53
29.00	0.506	7.071522	0.784159	2.352478	9.424	0.673783	51
30.00	0.524	6.544903	0.752865	2.258594	8.803497	0.783461	47
31.00	0.541	6.082699	0.723407	2.17022	8.252919	0.877833	44
32.00	0.559	5.673538	0.695612	2.086836	7.760374	0.959373	41
33.00	0.576	5.308574	0.669328	2.007984	7.316557	1.030029	38
34.00	0.593	4.980823	0.64442	1.933259	6.914082	1.091356	35
35.00	0.611	4.684698	0.620768	1.862305	6.547003	1.144615	32
36.00	0.628	4.415677	0.598267	1.794802	6.210479	1.190835	29
37.00	0.646	4.170057	0.576822	1.730466	5.900523	1.230868	26
38.00	0.663	3.944781	0.556348	1.669044	5.613825	1.265426	24
39.00	0.681	3.737301	0.536769	1.610306	5.347607	1.295106	21
40.00	0.698	3.545476	0.518016	1.554047	5.099522	1.320412	19
41.00	0.716	3.367493	0.500027	1.50008	4.867573	1.341776	16
42.00	0.733	3.20181	0.482746	1.448239	4.650048	1.359566	14
43.00	0.750	3.047102	0.466123	1.398369	4.445471	1.374098	12
44.00	0.768	2.902229	0.45011	1.350331	4.25256	1.385647	10
45.00	0.785	2.766201	0.434667	1.304	4.070201	1.39445	7
46.00	0.803	2.638158	0.419753	1.259258	3.897416	1.400713	5
47.00	0.820	2.517343	0.405333	1.216	3.733343	1.404616	3
48.00	0.838	2.403095	0.391376	1.174127	3.577222	1.406315	1
49.00	0.855	2.294828	0.37785	1.13355	3.428377	1.405947	2
50.00	0.873	2.192022	0.364729	1.094186	3.286208	1.403632	4
51.00	0.890	2.094218	0.351986	1.055958	3.150176	1.399473	6
52.00	0.908	2.001003	0.339599	1.018796	3.019799	1.393563	8
53.00	0.925	1.912009	0.327545	0.982634	2.894643	1.385978	9
54.00	0.942	1.826904	0.315804	0.947411	2.774315	1.376789	11
55.00	0.960	1.745391	0.304357	0.913071	2.658461	1.366053	13
56.00	0.977	1.667199	0.293186	0.879559	2.546758	1.353819	15
57.00	0.995	1.592087	0.282276	0.846827	2.438914	1.340131	17

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
58.00	1.012291	1.519831	0.27161	0.81483	2.33466	1.325023	18
59.00	1.029744	1.45023	0.261174	0.783522	2.233752	1.308521	20
60.00	1.047198	1.383101	0.250955	0.752865	2.135965	1.290649	22
61.00	1.064651	1.318275	0.24094	0.722819	2.041094	1.271422	23
62.00	1.082104	1.255598	0.231116	0.693349	1.948947	1.25085	25
63.00	1.099557	1.194929	0.221474	0.664421	1.85935	1.228937	27
64.00	1.117011	1.136138	0.212001	0.636003	1.772141	1.205684	28
65.00	1.134464	1.079103	0.202688	0.608065	1.687169	1.181085	30
66.00	1.151917	1.023716	0.193526	0.580578	1.604294	1.155129	31
67.00	1.169371	0.969873	0.184505	0.553515	1.523388	1.127802	33
68.00	1.186824	0.917478	0.175617	0.52685	1.444328	1.099083	34
69.00	1.204277	0.866444	0.166853	0.500559	1.367002	1.068947	36
70.00	1.22173	0.816688	0.158206	0.474617	1.291305	1.037365	37
71.00	1.239184	0.768133	0.149668	0.449003	1.217136	1.004302	39
72.00	1.256637	0.720709	0.141232	0.423695	1.144404	0.969716	40
73.00	1.27409	0.674347	0.132891	0.398673	1.07302	0.933564	42
74.00	1.291544	0.628986	0.124639	0.373916	1.002902	0.895792	43
75.00	1.308997	0.584567	0.116469	0.349406	0.933973	0.856344	45
76.00	1.32645	0.541034	0.108375	0.325124	0.866158	0.815155	46
77.00	1.343904	0.498336	0.100351	0.301052	0.799388	0.772156	48
78.00	1.361357	0.456422	0.092391	0.277174	0.733596	0.727269	49
79.00	1.37881	0.415248	0.084491	0.253472	0.66872	0.680407	50
80.00	1.396263	0.374769	0.076643	0.22993	0.604699	0.631477	52

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	48 °
Active earth pressure (P_A)	1.41 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{array}{l} P_{AH} = P_A \cos(\delta + \alpha) \\ P_{AV} = P_A \sin(\delta + \alpha) \end{array}\right\} \quad \cdots\cdots (7.1.15)$$

$$h_p = \frac{1}{3} H \quad \cdots\cdots (7.1.16)$$

where P_{AH}: Horizontal component of active earth pressure (kN/m)
P_{AV}: Vertical component of active earth pressure (kN/m)
h_p: Vertical distance to the action point of active earth pressure (m)
H: Height of wall (m)

P_{AH}=	1.35 KN/m
P_{AV}=	0.40 KN/m
h_p=	0.15 m

②Stability analysis (The case of dynamic)

1.Basic calculation

(1)Area

External force		Area			
Dead load;	W1	0.14m2=	0.30m×	0.45m	
	W2	0.00m2=	0.00m×	0.45m×	1/2
	W3	0.00m2=	0.00m×	0.45m×	1/2
Static water pressure;	P1	0.10m2=	0.45m×	0.45m×	1/2
Uplift;	U1	0.07m2=	0.45m×	0.30m×	1/2
Earth pressure	P_{AH}				
	P_{AV}				

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;	P1	0.15m=	0.45m×	1/3		
Uplift;	U1	0.10m=	0.30m×	1/3		
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)

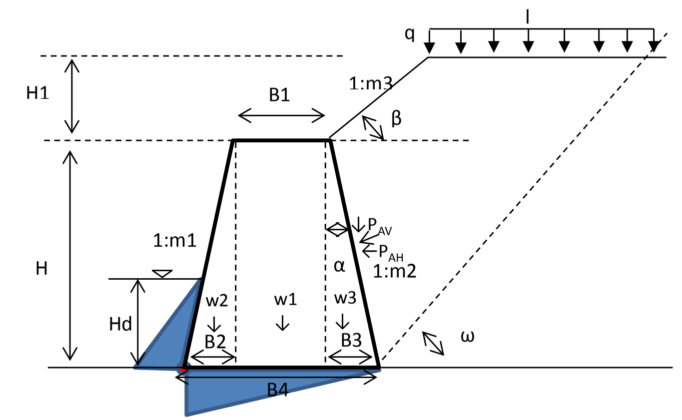
(3)External force by unit width

External force		External force by unit width			
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2	
	W2	0.00kN=	0.00m2×	23.00kN/m2	
	W3	0.00kN=	0.00m2×	23.00kN/m2	
Static water pressure;	P1	0.98kN=	0.10m2×	9.80kN/m2	
Uplift;	U1	-0.66kN=	0.07m2×	-9.80kN/m2	×1.0
Earth pressure;	P_{AH}	-1.35kN			
	P_{AV}	0.40kN			

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure	P1				0.98	0.15	0.15
Uplift	U1	-0.66	0.10	-0.07			
Earth pressure	P_{AH}				-1.35	0.15	-0.20
	P_{AV}	0.40	0.30	0.12			
Total		2.96		0.53	-0.37		-0.05



Type 2

①Calculation of earth pressure (The case of static)

1. Calculation of active earth pressure

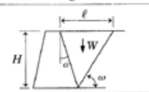
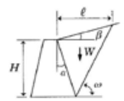
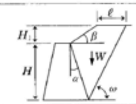
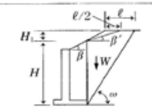
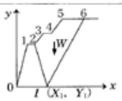
$$\frac{P_A}{\sin(\omega-\phi)} = \frac{W}{\sin\{90°-(\omega-\phi-\delta-\alpha)\}}$$

$$\therefore P_A = \frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)} W \quad \text{......(7.1.14)}$$

where P_A: Active earth pressure (kN/m)
W: Weight of soil wedge (including overburden load) (kN/m)
ϕ: Angle of shear resistance of backside ground (°)
α: Angle between wall's back surface and vertical plane (°) (Figure 7.1.10)
δ: Angle of wall friction (°) (Table 7.1.4)
ω: Angle between the assumed sliding surface and horizontal plane (°)

Figure 7.1.10 Illustrated examples of positive and negative values for α

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot H$
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha)\cdot\cos(\alpha-\beta)}{\cos^2\alpha \cdot \sin(\omega-\beta)}$ $\ell = \frac{\cos\beta \cdot \cos(\omega-\alpha)}{\cos\alpha \cdot \sin(\omega-\beta)} \cdot H$
	$W = \frac{1}{2}\gamma \left\{ (H+H_1)^2 \frac{\cos(\omega-\alpha)}{\sin\omega} - H_1{}^2 \frac{\cos(\alpha-\beta)}{\sin\beta} \right\} \frac{1}{\cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot (H+H_1) - \frac{\cos(\beta-\alpha)}{\sin\beta \cdot \cos\alpha} \cdot H_1$
	$W = \frac{1}{2}\gamma \{(H+H_1)^2\cot\omega - H_1{}^2\cot\beta\}$ $\beta' = \tan^{-1}\frac{2H_1}{(H+H_1)\cot\omega + H_1\cot\beta}$ $\ell = (H+H_1)\cot\omega - H_1\cot\beta$
	$W = \frac{1}{2}\gamma \sum_{i=1}^{n} (X_{i+1}-X_i)(Y_{i+1}+Y_i)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	10.20879	2.26862	6.80586	17.01465	0	56
26.00	0.454	9.213617	2.408933	7.226799	16.44042	0.297996	55
27.00	0.471	8.382446	1.546812	4.640436	13.02288	0.4698	54
28.00	0.489	7.677427	0.817489	2.452467	10.12989	0.545606	53
29.00	0.506	7.071522	0.784159	2.352478	9.424	0.673783	51
30.00	0.524	6.544903	0.752865	2.258594	8.803497	0.783461	47
31.00	0.541	6.082699	0.723407	2.17022	8.252919	0.877833	44
32.00	0.559	5.673538	0.695612	2.086836	7.760374	0.959373	41
33.00	0.576	5.308574	0.669328	2.007984	7.316557	1.030029	38
34.00	0.593	4.980823	0.64442	1.933259	6.914082	1.091356	35
35.00	0.611	4.684698	0.620768	1.862305	6.547003	1.144615	32
36.00	0.628	4.415677	0.598267	1.794802	6.210479	1.190835	29
37.00	0.646	4.170057	0.576822	1.730466	5.900523	1.230868	26
38.00	0.663	3.944781	0.556348	1.669044	5.613825	1.265426	24
39.00	0.681	3.737301	0.536769	1.610306	5.347607	1.295106	21
40.00	0.698	3.545476	0.518016	1.554047	5.099522	1.320412	19
41.00	0.716	3.367493	0.500027	1.50008	4.867573	1.341776	16
42.00	0.733	3.20181	0.482746	1.448239	4.650048	1.359566	14
43.00	0.750	3.047102	0.466123	1.398369	4.445471	1.374098	12
44.00	0.768	2.902229	0.45011	1.350331	4.25256	1.385647	10
45.00	0.785	2.766201	0.434667	1.304	4.070201	1.39445	7
46.00	0.803	2.638158	0.419753	1.259258	3.897416	1.400713	5
47.00	0.820	2.517343	0.405333	1.216	3.733343	1.404616	3
48.00	0.838	2.403095	0.391376	1.174127	3.577222	1.406315	1
49.00	0.855	2.294828	0.37785	1.13355	3.428377	1.405947	2
50.00	0.873	2.192022	0.364729	1.094186	3.286208	1.403632	4
51.00	0.890	2.094218	0.351986	1.055958	3.150176	1.399473	6
52.00	0.908	2.001003	0.339599	1.018796	3.019799	1.393563	8
53.00	0.925	1.912009	0.327545	0.982634	2.894643	1.385978	9
54.00	0.942	1.826904	0.315804	0.947411	2.774315	1.376789	11
55.00	0.960	1.745391	0.304357	0.913071	2.658461	1.366053	13
56.00	0.977	1.667199	0.293186	0.879559	2.546758	1.353819	15
57.00	0.995	1.592087	0.282276	0.846827	2.438914	1.340131	17

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
58.00	1.012291	1.519831	0.27161	0.81483	2.33466	1.325023	18
59.00	1.029744	1.45023	0.261174	0.783522	2.233752	1.308521	20
60.00	1.047198	1.383101	0.250955	0.752865	2.135965	1.290649	22
61.00	1.064651	1.318275	0.24094	0.722819	2.041094	1.271422	23
62.00	1.082104	1.255598	0.231116	0.693349	1.948947	1.25085	25
63.00	1.099557	1.194929	0.221474	0.664421	1.85935	1.228937	27
64.00	1.117011	1.136138	0.212001	0.636003	1.772141	1.205684	28
65.00	1.134464	1.079103	0.202688	0.608065	1.687169	1.181085	30
66.00	1.151917	1.023716	0.193526	0.580578	1.604294	1.155129	31
67.00	1.169371	0.969873	0.184505	0.553515	1.523388	1.127802	33
68.00	1.186824	0.917478	0.175617	0.52685	1.444328	1.099083	34
69.00	1.204277	0.866444	0.166853	0.500559	1.367002	1.068947	36
70.00	1.22173	0.816688	0.158206	0.474617	1.291305	1.037365	37
71.00	1.239184	0.768133	0.149668	0.449003	1.217136	1.004302	39
72.00	1.256637	0.720709	0.141232	0.423695	1.144404	0.969716	40
73.00	1.27409	0.674347	0.132891	0.398673	1.07302	0.933564	42
74.00	1.291544	0.628986	0.124639	0.373916	1.002902	0.895792	43
75.00	1.308997	0.584567	0.116469	0.349406	0.933973	0.856344	45
76.00	1.32645	0.541034	0.108375	0.325124	0.866158	0.815155	46
77.00	1.343904	0.498336	0.100351	0.301052	0.799388	0.772156	48
78.00	1.361357	0.456422	0.092391	0.277174	0.733596	0.727269	49
79.00	1.37881	0.415248	0.084491	0.253472	0.66872	0.680407	50
80.00	1.396263	0.374769	0.076643	0.22993	0.604699	0.631477	52

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	48 °
Active earth pressure (P_A)	1.41 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{array}{l} P_{AH} = P_A \cos(\delta + \alpha) \\ P_{AV} = P_A \sin(\delta + \alpha) \end{array}\right\} \quad \cdots\cdots (7.1.15)$$

$$h_p = \frac{1}{3} H \quad \cdots\cdots (7.1.16)$$

where
P_{AH}: Horizontal component of active earth pressure (kN/m)
P_{AV}: Vertical component of active earth pressure (kN/m)
h_p: Vertical distance to the action point of active earth pressure (m)
H: Height of wall (m)

P_{AH}=	1.35 KN/m
P_{AV}=	0.40 KN/m
h_p=	0.15 m

②Stability analysis (The case of static)

1.Basic calculation

(1)Area

External force		Area				
Dead load;	W1	0.14m2=	0.30m×	0.45m		
	W2	0.00m2=	0.00m×	0.45m×	1/2	
	W3	0.00m2=	0.00m×	0.45m×	1/2	
Static water pressure;						
Uplift;						
Earth pressure	P_{AH}					
	P_{AV}					

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;						
Uplift;						
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)

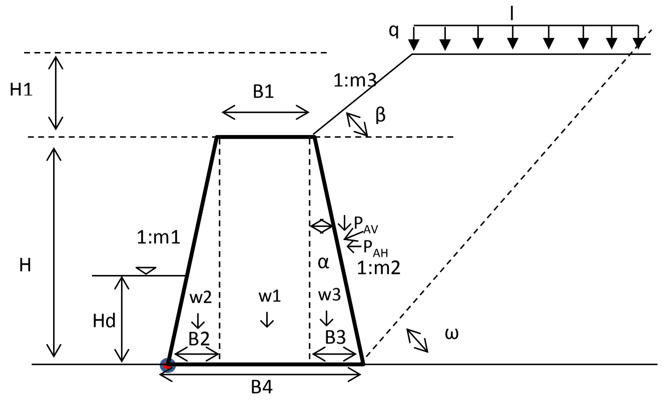
(3)External force by unit width

External force		External force by unit width		
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2
	W2	0.00kN=	0.00m2×	23.00kN/m2
	W3	0.00kN=	0.00m2×	23.00kN/m2
Static water pressure;				
Uplift;				
Earth pressure;	P_{AH}	-1.35kN		
	P_{AV}	0.40kN		

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure							
Uplift							
Earth pressure	P_{AH}				-1.35	0.15	-0.20
	P_{AV}	0.40	0.30	0.12			
Total		3.62		0.60	-1.35		-0.20



Type 2

①Calculation of earth pressure (The case of earthquick)

1. Calculation of active earth pressure

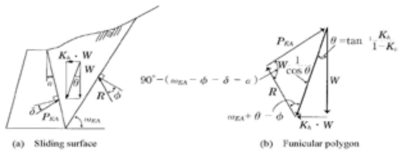


Figure 7.1.15 Concept of active earth pressure under seismic condition

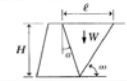
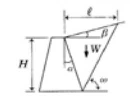
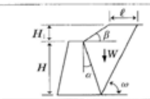
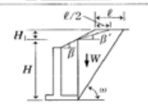
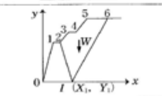
$$P_{EA}=\frac{\sin(\omega_{EA}-\phi+\theta)}{\cos(\omega_{EA}-\phi-\delta-\alpha)\cdot\cos\theta}W \quad \text{.......(7.1.18)}$$

$$\left.\begin{aligned}P_{EAH}&=P_{EA}\cos(\delta+\alpha)\\P_{EAV}&=P_{EA}\sin(\delta+\alpha)\end{aligned}\right\} \quad \text{.......(7.1.19)}$$

$$h_p=\frac{1}{3}H \quad \text{.......(7.1.20)}$$

where
- P_{EA}: Active earth pressure under seismic condition (kN/m)
- P_{EAH}: Horizontal component of active earth pressure under seismic condition (kN/m)
- P_{EAV}: Vertical component of active earth pressure under seismic condition (kN/m)
- ω_{EA}: Angle between assumed sliding surface and horizontal plane (°)
- θ: Seismic compound angle ($\tan^{-1}\frac{K_h}{1-K_v}$)(°)
- K_h: Design horizontal seismic coefficient (see the equation (7.1.44))
- δ: Angle of wall friction under seismic condition (°) (see the equation (7.1.4))
- h_p: Vertical distance to the action point of active earth pressure under seismic condition (m)

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot H$
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)\cdot\cos(\alpha-\beta)}{\cos^2\alpha\cdot\sin(\omega-\beta)}$ $\ell=\frac{\cos\beta\cdot\cos(\omega-\alpha)}{\cos\alpha\cdot\sin(\omega-\beta)}\cdot H$
	$W=\frac{1}{2}\gamma\left\{(H+H_1)^2\frac{\cos(\omega-\alpha)}{\sin\omega}-H_1^2\frac{\cos(\alpha-\beta)}{\sin\beta}\right\}\frac{1}{\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot(H+H_1)-\frac{\cos(\beta-\alpha)}{\sin\beta\cdot\cos\alpha}\cdot H_1$
	$W=\frac{1}{2}\gamma\{(H+H_1)^2\cot\omega-H_1^2\cot\beta\}$ $\beta'=\tan^{-1}\frac{2H_1}{(H+H_1)\cot\omega+H_1\cot\beta}$ $\ell=(H+H_1)\cot\omega-H_1\cot\beta$
	$W=\frac{1}{2}\gamma\sum_{i=1}^{n}(X_{i+1}-X_i)(Y_{i+1}+Y_i)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	10.20879	2.26862	6.80586	17.01465	0	56
26.00	0.454	9.213617	2.408933	7.226799	16.44042	0.292803	55
27.00	0.471	8.382446	1.546812	4.640436	13.02288	0.462232	54
28.00	0.489	7.677427	0.817489	2.452467	10.12989	0.53753	53
29.00	0.506	7.071522	0.784159	2.352478	9.424	0.664686	52
30.00	0.524	6.544903	0.752865	2.258594	8.803497	0.773896	48
31.00	0.541	6.082699	0.723407	2.17022	8.252919	0.868246	45
32.00	0.559	5.673538	0.695612	2.086836	7.760374	0.950126	42
33.00	0.576	5.308574	0.669328	2.007984	7.316557	1.021417	39
34.00	0.593	4.980823	0.64442	1.933259	6.914082	1.083622	36
35.00	0.611	4.684698	0.620768	1.862305	6.547003	1.137958	34
36.00	0.628	4.415677	0.598267	1.794802	6.210479	1.185421	31
37.00	0.646	4.170057	0.576822	1.730466	5.900523	1.226835	28
38.00	0.663	3.944781	0.556348	1.669044	5.613825	1.262884	26
39.00	0.681	3.737301	0.536769	1.610306	5.347607	1.294147	23
40.00	0.698	3.545476	0.518016	1.554047	5.099522	1.321111	21
41.00	0.716	3.367493	0.500027	1.50008	4.867573	1.344192	18
42.00	0.733	3.20181	0.482746	1.448239	4.650048	1.363746	16
43.00	0.750	3.047102	0.466123	1.398369	4.445471	1.38008	14
44.00	0.768	2.902229	0.45011	1.350331	4.25256	1.393456	12
45.00	0.785	2.766201	0.434667	1.304	4.070201	1.404103	9
46.00	0.803	2.638158	0.419753	1.259258	3.897416	1.412221	7
47.00	0.820	2.517343	0.405333	1.216	3.733343	1.417982	5
48.00	0.838	2.403095	0.391376	1.174127	3.577222	1.421536	3
49.00	0.855	2.294828	0.37785	1.13355	3.428377	1.423014	1
50.00	0.873	2.192022	0.364729	1.094186	3.286208	1.422531	2
51.00	0.890	2.094218	0.351986	1.055958	3.150176	1.420186	4
52.00	0.908	2.001003	0.339599	1.018796	3.019799	1.416065	6
53.00	0.925	1.912009	0.327545	0.982634	2.894643	1.410242	8
54.00	0.942	1.826904	0.315804	0.947411	2.774315	1.402782	10
55.00	0.960	1.745391	0.304357	0.913071	2.658461	1.393737	11
56.00	0.977	1.667199	0.293186	0.879559	2.546758	1.383154	13
57.00	0.995	1.592087	0.282276	0.846827	2.438914	1.37107	15
58.00	1.012	1.519831	0.27161	0.81483	2.33466	1.357516	17
59.00	1.030	1.45023	0.261174	0.783522	2.233752	1.342514	19
60.00	1.047	1.383101	0.250955	0.752865	2.135965	1.326081	20
61.00	1.065	1.318275	0.24094	0.722819	2.041094	1.308229	22
62.00	1.082	1.255598	0.231116	0.693349	1.948947	1.288962	24
63.00	1.100	1.194929	0.221474	0.664421	1.85935	1.268279	25
64.00	1.117	1.136138	0.212001	0.636003	1.772141	1.246174	27

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
65.00	1.134464	1.079103	0.202688	0.608065	1.687169	1.222636	29
66.00	1.151917	1.023716	0.193526	0.580578	1.604294	1.197646	30
67.00	1.169371	0.969873	0.184505	0.553515	1.523388	1.171182	32
68.00	1.186824	0.917478	0.175617	0.52685	1.444328	1.143217	33
69.00	1.204277	0.866444	0.166853	0.500559	1.367002	1.113717	35
70.00	1.22173	0.816688	0.158206	0.474617	1.291305	1.082641	37
71.00	1.239184	0.768133	0.149668	0.449003	1.217136	1.049945	38
72.00	1.256637	0.720709	0.141232	0.423695	1.144404	1.015578	40
73.00	1.27409	0.674347	0.132891	0.398673	1.07302	0.979479	41
74.00	1.291544	0.628986	0.124639	0.373916	1.002902	0.941586	43
75.00	1.308997	0.584567	0.116469	0.349406	0.933973	0.901824	44
76.00	1.32645	0.541034	0.108375	0.325124	0.866158	0.860113	46
77.00	1.343904	0.498336	0.100351	0.301052	0.799388	0.816363	47
78.00	1.361357	0.456422	0.092391	0.277174	0.733596	0.770478	49
79.00	1.37881	0.415248	0.084491	0.253472	0.66872	0.722347	50
80.00	1.396263	0.374769	0.076643	0.22993	0.604699	0.671851	51

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	49 °
Active earth pressure (P_A)	1.42 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{array}{l} P_{AH} = P_A \cos(\delta + \alpha) \\ P_{AV} = P_A \sin(\delta + \alpha) \end{array}\right\} \quad \cdots\cdots (7.1.15)$$

$$h_p = \frac{1}{3} H \quad \cdots\cdots (7.1.16)$$

where
- P_{AH}: Horizontal component of active earth pressure (kN/m)
- P_{AV}: Vertical component of active earth pressure (kN/m)
- h_p: Vertical distance to the action point of active earth pressure (m)
- H: Height of wall (m)

P_{AH}=	1.39 KN/m
P_{AV}=	0.31 KN/m
h_p=	0.15 m

②Stability analysis (The case of earthquick)

1.Basic calculation

(1)Area

External force		Area				
Dead load;	W1	0.14m2=	0.30m×	0.45m		
	W2	0.00m2=	0.00m×	0.45m×	1/2	
	W3	0.00m2=	0.00m×	0.45m×	1/2	
Static water pressure;						
Uplift;						
Earth pressure	P_{AH}					
	P_{AV}					

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;						
Uplift;						
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)
	S1	0.23m=	0.45m×	1/2		
	S2	0.15m=	0.45m×	1/3		
	S3	0.15m=	0.45m×	1/3		

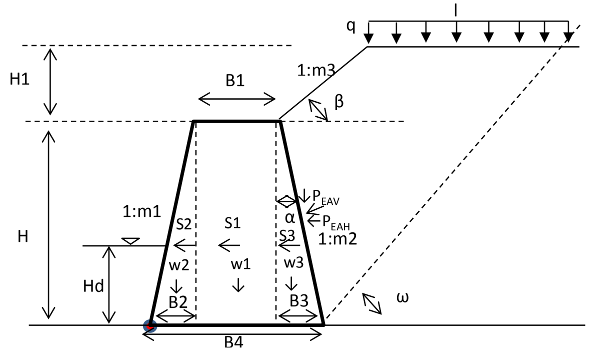
(3)External force by unit width

External force		External force by unit width		
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2
	W2	0.00kN=	0.00m2×	23.00kN/m2
	W3	0.00kN=	0.00m2×	23.00kN/m2
Static water pressure;				
Uplift;				
Earth pressure;	P_{AH}	-1.39kN		
	P_{AV}	0.31kN		
	S1	0.00kN=	-3.22KN×	0
	S2	0.00kN=	0.00KN×	0
	S3	0.00kN=	0.00KN×	0

2.Calculation table

(unit width)

		Vertical force V (kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure	P1						
Uplift	U1						
Earth pressure	P_{AH}				-1.39	0.15	-0.21
	P_{AV}	0.31	0.30	0.09			
	S1				0.00	0.23	0.00
	S2				0.00	0.15	0.00
	S3				0.00	0.15	0.00
Total		3.53		0.57	-1.39		-0.21



Excel format guide 2. Wall Stability Analysis Type3 (Summary)

#Please input data into yellow cell

1.Basic data for calculation

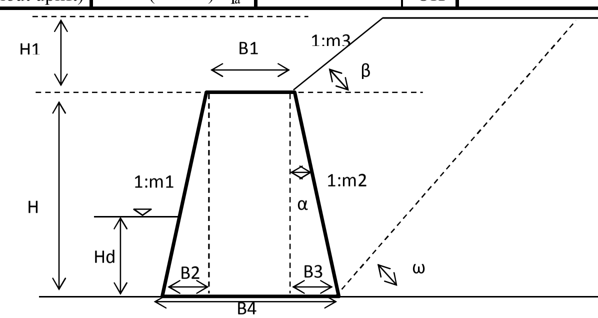
Item		Value			Reference
Unit weight of plane concrete	γc=	23	kN/m^3		Reinforced concrete: 24.5KN/m3, Plain concrete: 23KN/m3, Cement mortar: 21KN/m3
Unit weight of soil	γs=	20	kN/m^3		
internal friction angle	φ=	25	°	=0.436rad	
Angle between wall's back surface and vertical plane	α=	0.0	°	=0.000rad	
Inclination angle of wall's backside ground	β=	15	°	=0.262rad	
Angle of wall friction (normal condition)	δ=	16.7	°	=0.291rad	
Angle of wall friction (seismic condition)	δ=	12.5	°	=0.218rad	
Crowd load	q=	3.00	kN/m^2		
Unit weight of water	W_0=	9.8	kN/m^3		
Coefficient of earth pressure	C_0=	0.45			
Uplift coefficient	μ=	1.0			Rock foundation case or a case using sheet piles reaching an impermeable stratum: 0.4, Otherwise: 1.0
Seismic horizontal acceleration	kh=	0.15			
Seismic vertical acceleration	kv=	0			
Seismic compound angle	θ=	8.53	°	=0.149rad	
Friction coefficient	f=	0.7			
Allowable stress of the graund	qa=	50	kN/m^2		Bedrock=1000KN/m2 See 3.2.4

2.Mesurement of structure

Item		Value	
Slope of front body	m1 = 1 :	0.00	
Slope of back body	m2 = 1 :	0.00	
B1		0.30	m
B2		0.00	m
B3		0.00	m
B4 (B1+B2+B3)		0.30	m
H		0.45	m
H1		0.50	m
H_d		0.45	m

3. The result of calculation

Content		Dynamic				Static				Earthquake			
Overturning	e<B/6 or B/3	0.05	≤	0.05	OK	0.02	≤	0.05	OK	0.08	≤	0.10	OK
Sliding	ΣV･f/ΣH≥1.5or1.2	3.89	≥	1.50	OK	4.78	≥	1.50	OK	1.21	≥	1.20	OK
Settlement	ΣV/B･(1+6e/B)<q_a	17kN/m2	<	50kN/m2	OK	15kN/m2	<	50kN/m2	OK	32kN/m2	<	50kN/m2	OK
	ΣV/B･(1-6e/B)<q_a	1kN/m2	<	50kN/m2	OK	8kN/m2	<	50kN/m2	OK	-8kN/m2	<	50kN/m2	OK
(Without uplift)	ΣV/B･(1+6e/B)<q_a	18kN/m2	<	50kN/m2	OK								
(Without uplift)	ΣV/B･(1-6e/B)<q_a	5kN/m2	<	50kN/m2	OK								



Type 3

①Calculation of earth pressure (The case of dynamic)

1. Calculation of active earth pressure

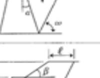
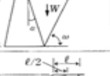
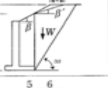
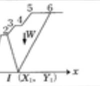
$$\frac{P_A}{\sin(\omega-\phi)}=\frac{W}{\sin\{90^\circ-(\omega-\phi-\delta-\alpha)}$$

$$\therefore P_A=\frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)}W \quad\text{.......................................}(7.1.14)$$

where P_A: Active earth pressure (kN/m)
W: Weight of soil wedge (including overburden load) (kN/m)
ϕ: Angle of shear resistance of backside ground (°)
α: Angle between wall's back surface and vertical plane (°) (Figure 7.1.10)
δ: Angle of wall friction (°) (Table 7.1.4)
ω: Angle between the assumed sliding surface and horizontal plane (°)

Figure 7.1.10 Illustrated examples of positive and negative values for α

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot H$
	$W=\frac{1}{2}\gamma\cdot H^2\frac{\cos(\omega-\alpha)\cdot\cos(\alpha-\beta)}{\cos^2\alpha\cdot\sin(\omega-\beta)}$ $\ell=\frac{\cos\beta\cdot\cos(\omega-\alpha)}{\cos\alpha\cdot\sin(\omega-\beta)}\cdot H$
	$W=\frac{1}{2}\gamma\left\{(H+H_1)^2\frac{\cos(\omega-\alpha)}{\sin\omega}-H_1{}^2\frac{\cos(\alpha-\beta)}{\sin\beta}\right\}\frac{1}{\cos\alpha}$ $\ell=\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha}\cdot(H+H_1)-\frac{\cos(\beta-\alpha)}{\sin\beta\cdot\cos\alpha}\cdot H_1$
	$W=\frac{1}{2}\gamma\{(H+H_1)^2\cot\omega-H_1{}^2\cot\beta\}$ $\beta'=\tan^{-1}\frac{2H_1}{(H+H_1)\cot\omega+H_1\cot\beta}$ $\ell=(H+H_1)\cot\omega-H_1\cot\beta$
	$W=\frac{1}{2}\gamma\sum_{i=1}^{n}(X_{i+1}-X_i)(Y_{i+1}+Y_i)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	10.02405	0.171256	0.513769	10.53782	0	14
26.00	0.454	9.173865	0.081763	0.24529	9.419155	0.17073	12
27.00	0.471	8.382433	-0.00155	-0.00464	8.377797	0.302229	9
28.00	0.489	7.643429	-0.07934	-0.23801	7.405424	0.398863	7
29.00	0.506	6.951404	-0.15218	-0.45654	6.494864	0.46436	5
30.00	0.524	6.301632	-0.22058	-0.66173	5.6399	0.501919	3
31.00	0.541	5.689995	-0.28496	-0.85488	4.835116	0.514294	1
32.00	0.559	5.112892	-0.34571	-1.03712	4.075769	0.503866	2
33.00	0.576	4.567154	-0.40315	-1.20946	3.357693	0.472698	4
34.00	0.593	4.049986	-0.45759	-1.37278	2.677208	0.422585	6
35.00	0.611	3.558909	-0.50928	-1.52785	2.031054	0.35509	8
36.00	0.628	3.09172	-0.55846	-1.67539	1.416332	0.271576	10
37.00	0.646	2.646452	-0.60533	-1.816	0.830454	0.173235	11
38.00	0.663	2.221346	-0.65008	-1.95024	0.271104	0.06111	13
39.00	0.681	1.81482	-0.69287	-2.07862	-0.2638	-0.06389	15
40.00	0.698	1.425449	-0.73386	-2.20158	-0.77613	-0.20096	16
41.00	0.716	1.051948	-0.77318	-2.31953	-1.26758	-0.34942	17
42.00	0.733	0.693151	-0.81094	-2.43283	-1.73968	-0.50864	18
43.00	0.750	0.348001	-0.84728	-2.54183	-2.19382	-0.67811	19
44.00	0.768	0.015534	-0.88227	-2.64681	-2.63128	-0.85737	20
45.00	0.785	-0.30513	-0.91603	-2.74808	-3.0532	-1.04603	21
46.00	0.803	-0.61479	-0.94862	-2.84586	-3.46065	-1.24374	22
47.00	0.820	-0.91418	-0.98014	-2.94041	-3.85459	-1.45023	23
48.00	0.838	-1.20398	-1.01064	-3.03192	-4.23591	-1.66526	24
49.00	0.855	-1.48481	-1.0402	-3.12061	-4.60542	-1.88864	25
50.00	0.873	-1.75725	-1.06888	-3.20664	-4.9639	-2.12022	26
51.00	0.890	-2.02183	-1.09673	-3.29019	-5.31202	-2.35988	27
52.00	0.908	-2.27902	-1.1238	-3.37141	-5.65044	-2.60754	28
53.00	0.925	-2.5293	-1.15015	-3.45045	-5.97975	-2.86315	29
54.00	0.942	-2.77308	-1.17581	-3.52743	-6.30051	-3.12671	30
55.00	0.960	-3.01075	-1.20083	-3.60248	-6.61324	-3.39822	31
56.00	0.977	-3.24269	-1.22524	-3.67573	-6.91841	-3.67773	32
57.00	0.995	-3.46922	-1.24909	-3.74726	-7.21649	-3.96531	33

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
58.00	1.012291	-3.69068	-1.2724	-3.8172	-7.50788	-4.26105	34
59.00	1.029744	-3.90736	-1.29521	-3.88562	-7.79298	-4.56509	35
60.00	1.047198	-4.11954	-1.31754	-3.95263	-8.07217	-4.87758	36
61.00	1.064651	-4.32749	-1.33943	-4.0183	-8.34578	-5.19869	37
62.00	1.082104	-4.53145	-1.3609	-4.0827	-8.61415	-5.52863	38
63.00	1.099557	-4.73166	-1.38198	-4.14593	-8.87759	-5.86764	39
64.00	1.117011	-4.92834	-1.40268	-4.20804	-9.13638	-6.21598	40
65.00	1.134464	-5.1217	-1.42303	-4.2691	-9.3908	-6.57393	41
66.00	1.151917	-5.31194	-1.44306	-4.32917	-9.64111	-6.94182	42
67.00	1.169371	-5.49924	-1.46277	-4.38832	-9.88756	-7.32001	43
68.00	1.186824	-5.68379	-1.4822	-4.4466	-10.1304	-7.70887	44
69.00	1.204277	-5.86575	-1.50135	-4.50406	-10.3698	-8.10883	45
70.00	1.22173	-6.0453	-1.52025	-4.56076	-10.6061	-8.52034	46
71.00	1.239184	-6.22257	-1.53891	-4.61674	-10.8393	-8.9439	47
72.00	1.256637	-6.39773	-1.55735	-4.67206	-11.0698	-9.38004	48
73.00	1.27409	-6.57091	-1.57558	-4.72674	-11.2977	-9.82934	49
74.00	1.291544	-6.74225	-1.59362	-4.78085	-11.5231	-10.2924	50
75.00	1.308997	-6.91189	-1.61147	-4.83442	-11.7463	-10.77	51
76.00	1.32645	-7.07994	-1.62916	-4.88749	-11.9674	-11.2627	52
77.00	1.343904	-7.24654	-1.6467	-4.9401	-12.1866	-11.7715	53
78.00	1.361357	-7.4118	-1.6641	-4.99229	-12.4041	-12.2971	54
79.00	1.37881	-7.57584	-1.68136	-5.04409	-12.6199	-12.8405	55
80.00	1.396263	-7.73878	-1.69851	-5.09554	-12.8343	-13.4027	56

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	31 °
Active earth pressure (P_A)	0.51 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{array}{l} P_{AH} = P_A \cos(\delta + \alpha) \\ P_{AV} = P_A \sin(\delta + \alpha) \end{array}\right\} \quad \text{(7.1.15)}$$

$$h_p = \frac{1}{3} H \quad \text{(7.1.16)}$$

where P_{AH}: Horizontal component of active earth pressure (kN/m)
P_{AV}: Vertical component of active earth pressure (kN/m)
h_p: Vertical distance to the action point of active earth pressure (m)
H: Height of wall (m)

P_{AH}=	0.49 KN/m
P_{AV}=	0.15 KN/m
h_p=	0.15 m

②Stability analysis (The case of dynamic)

1.Basic calculation

(1)Area

External force		Area				
Dead load;	W1	0.14m2=	0.30m×	0.45m		
	W2	0.00m2=	0.00m×	0.45m×	1/2	
	W3	0.00m2=	0.00m×	0.45m×	1/2	
Static water pressure;	P1	0.10m2=	0.45m×	0.45m×	1/2	
Uplift;	U1	0.07m2=	0.45m×	0.30m×	1/2	
Earth pressure	P_{AH}					
	P_{AV}					

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;	P1	0.15m=	0.45m×	1/3		
Uplift;	U1	0.10m=	0.30m×	1/3		
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)

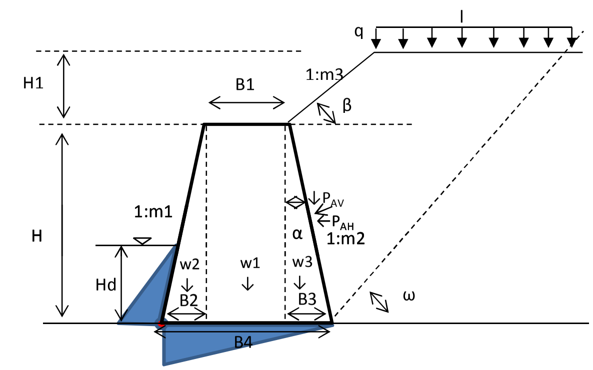
(3)External force by unit width

External force		External force by unit width			
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2	
	W2	0.00kN=	0.00m2×	23.00kN/m2	
	W3	0.00kN=	0.00m2×	23.00kN/m2	
Static water pressure;	P1	0.98kN=	0.10m2×	9.80kN/m2	
Uplift;	U1	-0.66kN=	0.07m2×	-9.80kN/m2	×1.0
Earth pressure;	P_{AH}	-0.49kN			
	P_{AV}	0.15kN			

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure	P1				0.98	0.15	0.15
Uplift	U1	-0.66	0.10	-0.07			
Earth pressure	P_{AH}				-0.49	0.15	-0.07
	P_{AV}	0.15	0.30	0.04			
Total		2.71		0.45	0.49		0.08



Type 3

①Calculation of earth pressure (The case of static)

1. Calculation of active earth pressure

$$\frac{P_A}{\sin(\omega-\phi)} = \frac{W}{\sin\{90° - (\omega-\phi-\delta-\alpha)}$$

$$\therefore P_A = \frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)} W \qquad (7.1.14)$$

where P_A: Active earth pressure (kN/m)
W: Weight of soil wedge (including overburden load) (kN/m)
ϕ: Angle of shear resistance of backside ground (°)
α: Angle between wall's back surface and vertical plane (°) (Figure 7.1.10)
δ: Angle of wall friction (°) (Table 7.1.4)
ω: Angle between the assumed sliding surface and horizontal plane (°)

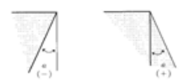
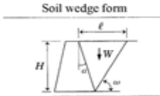
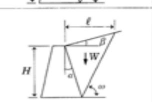
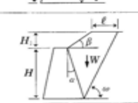
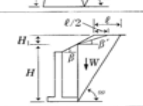
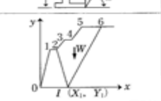


Figure 7.1.10 Illustrated examples of positive and negative values for α

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot H$
	$W = \frac{1}{2}\gamma \cdot H^2 \frac{\cos(\omega-\alpha) \cdot \cos(\alpha-\beta)}{\cos^2\alpha \cdot \sin(\omega-\beta)}$ $\ell = \frac{\cos\beta \cdot \cos(\omega-\alpha)}{\cos\alpha \cdot \sin(\omega-\beta)} \cdot H$
	$W = \frac{1}{2}\gamma \left\{ (H+H_1)^2 \frac{\cos(\omega-\alpha)}{\sin\omega} - H_1^2 \frac{\cos(\alpha-\beta)}{\sin\beta} \right\} \frac{1}{\cos\alpha}$ $\ell = \frac{\cos(\omega-\alpha)}{\sin\omega \cdot \cos\alpha} \cdot (H+H_1) - \frac{\cos(\beta-\alpha)}{\sin\beta \cdot \cos\alpha} \cdot H_1$
	$W = \frac{1}{2}\gamma \{(H+H_1)^2\cot\omega - H_1^2\cot\beta\}$ $\beta' = \tan^{-1}\frac{2H_1}{(H+H_1)\cot\omega + H_1\cot\beta}$ $\ell = (H+H_1)\cot\omega - H_1\cot\beta$
	$W = \frac{1}{2}\gamma \sum_{i=1}^{n} (X_{i+1} - X_i)(Y_{i+1} + Y_i)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	10.02405	0.171256	0.513769	10.53782	0	14
26.00	0.454	9.173865	0.081763	0.24529	9.419155	0.17073	12
27.00	0.471	8.382433	-0.00155	-0.00464	8.377797	0.302229	9
28.00	0.489	7.643429	-0.07934	-0.23801	7.405424	0.398863	7
29.00	0.506	6.951404	-0.15218	-0.45654	6.494864	0.46436	5
30.00	0.524	6.301632	-0.22058	-0.66173	5.6399	0.501919	3
31.00	0.541	5.689995	-0.28496	-0.85488	4.835116	0.514294	1
32.00	0.559	5.112892	-0.34571	-1.03712	4.075769	0.503866	2
33.00	0.576	4.567154	-0.40315	-1.20946	3.357693	0.472698	4
34.00	0.593	4.049986	-0.45759	-1.37278	2.677208	0.422585	6
35.00	0.611	3.558909	-0.50928	-1.52785	2.031054	0.35509	8
36.00	0.628	3.09172	-0.55846	-1.67539	1.416332	0.271576	10
37.00	0.646	2.646452	-0.60533	-1.816	0.830454	0.173235	11
38.00	0.663	2.221346	-0.65008	-1.95024	0.271104	0.06111	13
39.00	0.681	1.81482	-0.69287	-2.07862	-0.2638	-0.06389	15
40.00	0.698	1.425449	-0.73386	-2.20158	-0.77613	-0.20096	16
41.00	0.716	1.051948	-0.77318	-2.31953	-1.26758	-0.34942	17
42.00	0.733	0.693151	-0.81094	-2.43283	-1.73968	-0.50864	18
43.00	0.750	0.348001	-0.84728	-2.54183	-2.19382	-0.67811	19
44.00	0.768	0.015534	-0.88227	-2.64681	-2.63128	-0.85737	20
45.00	0.785	-0.30513	-0.91603	-2.74808	-3.0532	-1.04603	21
46.00	0.803	-0.61479	-0.94862	-2.84586	-3.46065	-1.24374	22
47.00	0.820	-0.91418	-0.98014	-2.94041	-3.85459	-1.45023	23
48.00	0.838	-1.20398	-1.01064	-3.03192	-4.23591	-1.66526	24
49.00	0.855	-1.48481	-1.0402	-3.12061	-4.60542	-1.88864	25
50.00	0.873	-1.75725	-1.06888	-3.20664	-4.9639	-2.12022	26
51.00	0.890	-2.02183	-1.09673	-3.29019	-5.31202	-2.35988	27
52.00	0.908	-2.27902	-1.1238	-3.37141	-5.65044	-2.60754	28
53.00	0.925	-2.5293	-1.15015	-3.45045	-5.97975	-2.86315	29
54.00	0.942	-2.77308	-1.17581	-3.52743	-6.30051	-3.12671	30
55.00	0.960	-3.01075	-1.20083	-3.60248	-6.61324	-3.39822	31
56.00	0.977	-3.24269	-1.22524	-3.67573	-6.91841	-3.67773	32
57.00	0.995	-3.46922	-1.24909	-3.74726	-7.21649	-3.96531	33

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
58.0	1.012291	-3.69068	-1.2724	-3.8172	-7.50788	-4.26105	34
59.0	1.029744	-3.90736	-1.29521	-3.88562	-7.79298	-4.56509	35
60.0	1.047198	-4.11954	-1.31754	-3.95263	-8.07217	-4.87758	36
61.0	1.064651	-4.32749	-1.33943	-4.0183	-8.34578	-5.19869	37
62.0	1.082104	-4.53145	-1.3609	-4.0827	-8.61415	-5.52863	38
63.0	1.099557	-4.73166	-1.38198	-4.14593	-8.87759	-5.86764	39
64.0	1.117011	-4.92834	-1.40268	-4.20804	-9.13638	-6.21598	40
65.0	1.134464	-5.1217	-1.42303	-4.2691	-9.3908	-6.57393	41
66.0	1.151917	-5.31194	-1.44306	-4.32917	-9.64111	-6.94182	42
67.0	1.169371	-5.49924	-1.46277	-4.38832	-9.88756	-7.32001	43
68.0	1.186824	-5.68379	-1.4822	-4.4466	-10.1304	-7.70887	44
69.0	1.204277	-5.86575	-1.50135	-4.50406	-10.3698	-8.10883	45
70.0	1.22173	-6.0453	-1.52025	-4.56076	-10.6061	-8.52034	46
71.0	1.239184	-6.22257	-1.53891	-4.61674	-10.8393	-8.9439	47
72.0	1.256637	-6.39773	-1.55735	-4.67206	-11.0698	-9.38004	48
73.0	1.27409	-6.57091	-1.57558	-4.72674	-11.2977	-9.82934	49
74.0	1.291544	-6.74225	-1.59362	-4.78085	-11.5231	-10.2924	50
75.0	1.308997	-6.91189	-1.61147	-4.83442	-11.7463	-10.77	51
76.0	1.32645	-7.07994	-1.62916	-4.88749	-11.9674	-11.2627	52
77.0	1.343904	-7.24654	-1.6467	-4.9401	-12.1866	-11.7715	53
78.0	1.361357	-7.4118	-1.6641	-4.99229	-12.4041	-12.2971	54
79.0	1.37881	-7.57584	-1.68136	-5.04409	-12.6199	-12.8405	55
80.0	1.396263	-7.73878	-1.69851	-5.09554	-12.8343	-13.4027	56

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	31 °
Active earth pressure (P_A)	0.51 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{aligned} P_{AH} &= P_A \cos(\delta + \alpha) \\ P_{AV} &= P_A \sin(\delta + \alpha) \end{aligned}\right\} \quad \text{.......... (7.1.15)}$$

$$h_p = \frac{1}{3}H \quad \text{.......... (7.1.16)}$$

where
- P_{AH}: Horizontal component of active earth pressure (kN/m)
- P_{AV}: Vertical component of active earth pressure (kN/m)
- h_p: Vertical distance to the action point of active earth pressure (m)
- H: Height of wall (m)

P_{AH}=	0.49 KN/m
P_{AV}=	0.15 KN/m
h_p=	0.15 m

②Stability analysis (The case of static)

1.Basic calculation

(1)Area

External force		Area				
Dead load;	W1	0.14m2=	0.30m×	0.45m		
	W2	0.00m2=	0.00m×	0.45m×	1/2	
	W3	0.00m2=	0.00m×	0.45m×	1/2	
Static water pressure;						
Uplift;						
Earth pressure	P_{AH}					
	P_{AV}					

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;						
Uplift;						
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)

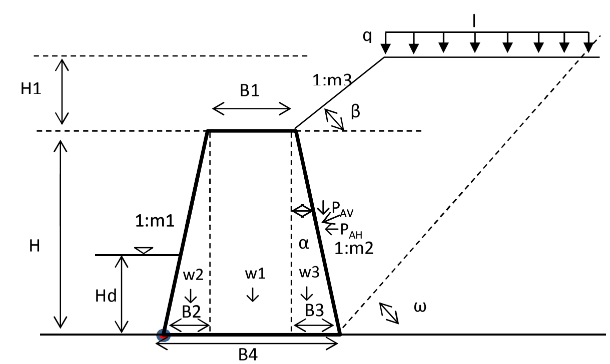
(3)External force by unit width

External force		External force by unit width		
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2
	W2	0.00kN=	0.00m2×	23.00kN/m2
	W3	0.00kN=	0.00m2×	23.00kN/m2
Static water pressure;				
Uplift;				
Earth pressure;	P_{AH}	-0.49kN		
	P_{AV}	0.15kN		

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x (m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y (m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure							
Uplift							
Earth pressure	P_{AH}				-0.49	0.15	-0.07
	P_{AV}	0.15	0.30	0.04			
Total		3.37		0.52	-0.49		-0.07



Type 3

①Calculation of earth pressure (The case of earthquick)

1. Calculation of active earth pressure

(a) Sliding surface (b) Funicular polygon

Figure 7.1.15 Concept of active earth pressure under seismic condition

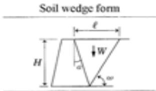
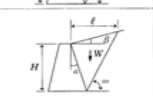
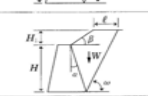
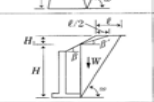
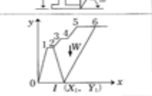
$$P_{EA} = \frac{\sin(\omega_{EA} - \phi + \theta)}{\cos(\omega_{EA} - \phi - \delta - \alpha) \cdot \cos\theta} W \quad \text{.......(7.1.18)}$$

$$\left.\begin{aligned} P_{EAH} &= P_{EA} \cos(\delta + \alpha) \\ P_{EAV} &= P_{EA} \sin(\delta + \alpha) \end{aligned}\right\} \quad \text{.......(7.1.19)}$$

$$h_p = \frac{1}{3} H \quad \text{.......(7.1.20)}$$

where P_{EA}: Active earth pressure under seismic condition (kN/m)
P_{EAH}: Horizontal component of active earth pressure under seismic condition (kN/m)
P_{EAV}: Vertical component of active earth pressure under seismic condition (kN/m)
ω_{EA}: Angle between assumed sliding surface and horizontal plane (°)
θ: Seismic compound angle ($\tan^{-1} \frac{K_h}{1-K_v}$) (°)
K_h: Design horizontal seismic coefficient (see the equation (7.1.44))
δ: Angle of wall friction under seismic condition (°) (see the equation (7.1.4))
h_p: Vertical distance to the action point of active earth pressure under seismic condition (m)

Table 7.1.7 Calculation formulas for soil wedge weight

Soil wedge form	Weight of soil wedge
	$W = \frac{1}{2} \gamma \cdot H^2 \frac{\cos(\omega - \alpha)}{\sin\omega \cdot \cos\alpha}$ $\ell = \frac{\cos(\omega - \alpha)}{\sin\omega \cdot \cos\alpha} \cdot H$
	$W = \frac{1}{2} \gamma \cdot H^2 \frac{\cos(\omega - \alpha) \cdot \cos(\alpha - \beta)}{\cos^2\alpha \cdot \sin(\omega - \beta)}$ $\ell = \frac{\cos\beta \cdot \cos(\omega - \alpha)}{\cos\alpha \cdot \sin(\omega - \beta)} \cdot H$
	$W = \frac{1}{2} \gamma \left\{ (H + H_1)^2 \frac{\cos(\omega - \alpha)}{\sin\omega} - H_1{}^2 \frac{\cos(\alpha - \beta)}{\sin\beta} \right\} \frac{1}{\cos\alpha}$ $\ell = \frac{\cos(\omega - \alpha)}{\sin\omega \cdot \cos\alpha} \cdot (H + H_1) - \frac{\cos(\beta - \alpha)}{\sin\beta \cdot \cos\alpha} \cdot H_1$
	$W = \frac{1}{2} \gamma \{(H + H_1)^2 \cot\omega - H_1{}^2 \cot\beta\}$ $\beta' = \tan^{-1} \frac{2 H_1}{(H + H_1)\cot\omega + H_1 \cot\beta}$ $\ell = (H + H_1)\cot\omega - H_1 \cot\beta$
	$W = \frac{1}{2} \gamma \sum_{i=1}^{n} (X_{i+1} - X_i)(Y_{i+1} + Y_i)$

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
25.00	0.436	10.02405	0.171256	0.513769	10.53782	1.61905	1
26.00	0.454	9.173865	0.081763	0.24529	9.419155	1.609353	2
27.00	0.471	8.382433	-0.00155	-0.00464	8.377797	1.574653	3
28.00	0.489	7.643429	-0.07934	-0.23801	7.405424	1.517675	4
29.00	0.506	6.951404	-0.15218	-0.45654	6.494864	1.44074	5
30.00	0.524	6.301632	-0.22058	-0.66173	5.6399	1.345829	6
31.00	0.541	5.689995	-0.28496	-0.85488	4.835116	1.234638	7
32.00	0.559	5.112892	-0.34571	-1.03712	4.075769	1.108624	8
33.00	0.576	4.567154	-0.40315	-1.20946	3.357693	0.96904	9
34.00	0.593	4.049986	-0.45759	-1.37278	2.677208	0.816969	10
35.00	0.611	3.558909	-0.50928	-1.52785	2.031054	0.65334	11
36.00	0.628	3.09172	-0.55846	-1.67539	1.416332	0.47896	12
37.00	0.646	2.646452	-0.60533	-1.816	0.830454	0.294518	13
38.00	0.663	2.221346	-0.65008	-1.95024	0.271104	0.100612	14
39.00	0.681	1.81482	-0.69287	-2.07862	-0.2638	-0.10225	15
40.00	0.698	1.425449	-0.73386	-2.20158	-0.77613	-0.31363	16
41.00	0.716	1.051948	-0.77318	-2.31953	-1.26758	-0.53316	17
42.00	0.733	0.693151	-0.81094	-2.43283	-1.73968	-0.76053	18
43.00	0.750	0.348001	-0.84728	-2.54183	-2.19382	-0.99548	19
44.00	0.768	0.015534	-0.88227	-2.64681	-2.63128	-1.23781	20
45.00	0.785	-0.30513	-0.91603	-2.74808	-3.0532	-1.48734	21
46.00	0.803	-0.61479	-0.94862	-2.84586	-3.46065	-1.74396	22
47.00	0.820	-0.91418	-0.98014	-2.94041	-3.85459	-2.00757	23
48.00	0.838	-1.20398	-1.01064	-3.03192	-4.23591	-2.27812	24
49.00	0.855	-1.48481	-1.0402	-3.12061	-4.60542	-2.55559	25
50.00	0.873	-1.75725	-1.06888	-3.20664	-4.9639	-2.83997	26
51.00	0.890	-2.02183	-1.09673	-3.29019	-5.31202	-3.13132	27
52.00	0.908	-2.27902	-1.1238	-3.37141	-5.65044	-3.42967	28
53.00	0.925	-2.5293	-1.15015	-3.45045	-5.97975	-3.73514	29
54.00	0.942	-2.77308	-1.17581	-3.52743	-6.30051	-4.04782	30
55.00	0.960	-3.01075	-1.20083	-3.60248	-6.61324	-4.36786	31
56.00	0.977	-3.24269	-1.22524	-3.67573	-6.91841	-4.69543	32
57.00	0.995	-3.46922	-1.24909	-3.74726	-7.21649	-5.0307	33
58.00	1.012	-3.69068	-1.2724	-3.8172	-7.50788	-5.3739	34
59.00	1.030	-3.90736	-1.29521	-3.88562	-7.79298	-5.72526	35
60.00	1.047	-4.11954	-1.31754	-3.95263	-8.07217	-6.08505	36
61.00	1.065	-4.32749	-1.33943	-4.0183	-8.34578	-6.45357	37
62.00	1.082	-4.53145	-1.3609	-4.0827	-8.61415	-6.83113	38
63.00	1.100	-4.73166	-1.38198	-4.14593	-8.87759	-7.21808	39
64.00	1.117	-4.92834	-1.40268	-4.20804	-9.13638	-7.61481	40

ω(°)	ω(rad)	W(KN/m3)	l(m)	Q=q*l	W+Q	P_A(KN/m)	Runking
65.00	1.134464	-5.1217	-1.42303	-4.2691	-9.3908	-8.02172	41
66.00	1.151917	-5.31194	-1.44306	-4.32917	-9.64111	-8.43927	42
67.00	1.169371	-5.49924	-1.46277	-4.38832	-9.88756	-8.86793	43
68.00	1.186824	-5.68379	-1.4822	-4.4466	-10.1304	-9.30823	44
69.00	1.204277	-5.86575	-1.50135	-4.50406	-10.3698	-9.76073	45
70.00	1.22173	-6.0453	-1.52025	-4.56076	-10.6061	-10.226	46
71.00	1.239184	-6.22257	-1.53891	-4.61674	-10.8393	-10.7048	47
72.00	1.256637	-6.39773	-1.55735	-4.67206	-11.0698	-11.1978	48
73.00	1.27409	-6.57091	-1.57558	-4.72674	-11.2977	-11.7056	49
74.00	1.291544	-6.74225	-1.59362	-4.78085	-11.5231	-12.2293	50
75.00	1.308997	-6.91189	-1.61147	-4.83442	-11.7463	-12.7695	51
76.00	1.32645	-7.07994	-1.62916	-4.88749	-11.9674	-13.3274	52
77.00	1.343904	-7.24654	-1.6467	-4.9401	-12.1866	-13.904	53
78.00	1.361357	-7.4118	-1.6641	-4.99229	-12.4041	-14.5003	54
79.00	1.37881	-7.57584	-1.68136	-5.04409	-12.6199	-15.1176	55
80.00	1.396263	-7.73878	-1.69851	-5.09554	-12.8343	-15.7573	56

The result of calculation

Angle between the sliding surface and horizontal plane (ω)	25 °
Active earth pressure (P_A)	1.62 KN/m

2. Horizontal, Vertical component and vertical distance to the action point of active earth pressure

$$\left.\begin{array}{l} P_{AH} = P_A \cos(\delta + \alpha) \\ P_{AV} = P_A \sin(\delta + \alpha) \end{array}\right\} \quad \cdots\cdots (7.1.15)$$

$$h_p = \frac{1}{3} H \quad \cdots\cdots (7.1.16)$$

where
- P_{AH}: Horizontal component of active earth pressure (kN/m)
- P_{AV}: Vertical component of active earth pressure (kN/m)
- h_p: Vertical distance to the action point of active earth pressure (m)
- H: Height of wall (m)

P_{AH}=	1.58 KN/m
P_{AV}=	0.35 KN/m
h_p=	0.15 m

②Stability analysis (The case of earthquick)

1.Basic calculation

(1)Area

External force		Area				
Dead load;	W1	0.14m2=	0.30m×	0.45m		
	W2	0.00m2=	0.00m×	0.45m×	1/2	
	W3	0.00m2=	0.00m×	0.45m×	1/2	
Static water pressure;						
Uplift;						
Earth pressure	P_{AH}					
	P_{AV}					

(2)Distance

External force		Distance				
Dead load;	W1	0.15m=	0.00m+	0.30m×	1/2	
	W2	0.00m=	0.00m×	2/3		
	W3	0.30m=	0.00m+	0.30m+	0.00m×	1/3
Static water pressure;						
Uplift;						
Earth pressure;	P_{AH}	0.15m=	0.45m×	1/3		
	P_{AV}	0.30m=	0.30m+	0.00m×	(0.45m-	0.15m)
	S1	0.23m=	0.45m×	1/2		
	S2	0.15m=	0.45m×	1/3		
	S3	0.15m=	0.45m×	1/3		

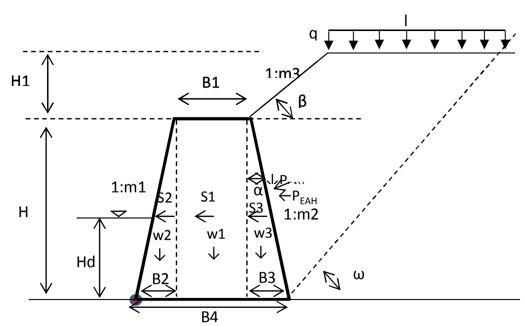
(3)External force by unit width

External force		External force by unit width		
Dead load;	W1	3.22kN=	0.14m2×	23.00kN/m2
	W2	0.00kN=	0.00m2×	23.00kN/m2
	W3	0.00kN=	0.00m2×	23.00kN/m2
Static water pressure;				
Uplift;				
Earth pressure;	P_{AH}	-1.58kN		
	P_{AV}	0.35kN		
	S1	-0.48kN=	-3.22KN×	0.15
	S2	0.00kN=	0.00KN×	0.15
	S3	0.00kN=	0.00KN×	0.15

2.Calculation table

(unit width)

		Vertical force V(kN)	Distance x(m)	Resistance moment V*x(kN·m)	Horizontal force H(kN)	Distance y(m)	Turning moment H*y(kN·m)
Dead load	W1	3.22	0.15	0.48			
	W2	0.00	0.00	0.00			
	W3	0.00	0.30	0.00			
water pressure	P1						
Uplift	U1						
Earth pressure	P_{AH}				-1.58	0.15	-0.24
	P_{AV}	0.35	0.30	0.11			
	S1				-0.48	0.23	-0.11
	S2				0.00	0.15	0.00
	S3				0.00	0.15	0.00
Total		3.57		0.59	-2.06		-0.35



5.2 Design of Related Structures

5.2.1 Design of Inner Space Width and Opening Height of Gate (Chapter 3.4.6)

Based on the assumed value of inner space width and opening height of gate, space width and opening height of gate can be calculated trial and error calculation. Refer to Excel format guide 3 for the calculation.
In case of free out flow ($Fr=V/\sqrt{(g\times h)} \geq 1$)

$$Q = Ca \times b \times d \times \sqrt{(2g \times h1)}$$

Division box	Gate	h1 (m)	h2 (m)	b (m)	D (m)	h1/d	h2/d	Ca	Q (m3/s)	Design max d(m)	d≤ Design d
DB1	Retaining wall canal → Secondary canal	0.24	0.053 ※1	0.2	0.14 ※2	1.74	0.38	0.45	0.027 ※1	0.45	OK
DB1	Retaining wall canal → Earthen canal	0.24	0.30	0.15	0.43 ※2	1.30	1.14	0.45	0.063	0.50	OK

※ 1; These value is calculated based on design of secondary canal

※ 2; These value is calculated by Trial and error calculation

Excel format guide 3. Opening height of gate (Division box)

1. Opening height of gate (d)

#Please input data into yellow cell
#The input data of green cell is the value assumed yourself. (Goal seek "By changing cell")
#Blue cell is the "set sell" for Goal seek. "to value" = Qd value

①Water depth upstream of gate (h1)	0.24	m	
②Water depth downstream of gate (h2)	0.053	m	
③Inner space width (b) =	0.2	m	
④Opening height of gate (d) =	0.138	m	← Goal seek ("By changing cell")
⑤h1 / d =	1.74		
⑥h2 / d =	0.38		
⑦Coefficient of discharge (Ca)	0.45		
⑧Discharge	0.027	m3/s	
⑨The result of calculation of discharge	0.027		← Goal seek ("set cell". "to value" = ⑧ value)

5.2.2 Design of Drops (Chapter 3.4.5)

(1) Basic data for design (in case of construction of drop into retaining wall canal)

・Design water intake discharge (Q) = 0.063m3/s,
・Upstream canal width (B1) = 0.5m,
Water depth (h1) = 0.24m, Velocity (V1) = 0.52m/s
・Downstream canal width (B2) = 0.5m,
Water depth (h2) = 0.24m, Velocity (V2) = 0.52m/s
・Width of outfall (B) = 0.5m (This value is assumption at first)
・Width of the water cushion (B0) = 0.7m (This value is assumption at first)
・Head of energy line to the center of water vain (Z) = 0.3
(This value is assumption at first)
・Depth of water cushion (h_D) = 0.15m (This value is assumption at first)

(2) Upstream canal ….. (See 3.4.5(3)2))
q = 0.063 / 0.5 =0.13 m3/s · m ≤ 2
So, L = 1.2 + 3√Q / 2 = 1.2 + 3√0.063 /2 = 1.58m ≈ 1.60m

(3) Critical Depth and others
Critical depth (hc), Specific energy at the critical depth of upstream canal (H), Head of energy line to the center of water vain (Z), Vertical distance measured downward from the outfall's downstream end (y) ….. (See 3.4.5(3)3)(c), 3.4.5(3)4)(d), 3.2.4(2))

$h_c = 0.467q^{2/3} = 0.467 \times 0.126^{2/3} = 0.12$m
$h_{vc} = (Q / (hc \times B))^2 / 2g = (0.063 / (0.12 \times 0.5))^2 / (2 \times 9.8) = 0.06$m
$H = 1.5 \times hc = 1.5 \times 0.12 = 0.18$m
$Z = y + H = y + 0.18 = 0.3$
$\therefore\ y = 0.3 - 0.18 = 0.12$ m

(4) Shape of water vein ….. (See 3.4.5(3)4)(a))
In case of with afflux, the calculation is as follows;
(a) Track of the center of water vein

$X/H = 1.155 \{ (y/H) + 0.333 \}^{0.5}$
$\therefore\ X = 1.155 \times 0.18 \{ (0.12/0.18) + 0.333 \}^{0.5} = 0.21$m

(b) Inclination angle of water vein

$\tan\theta = 1.5\ (X/H) = 1.5\ (0.21/0.18) = 1.75$

$\therefore \theta = 60.3°$

(c) Velocity at the center of water vein

$V = \sqrt{(2gz)} = \sqrt{(2 \times 9.8 \times 0.3)} = 2.42m/s$

(d) Thickness of water vein

$d = Q\ /\ (B \times V) = 0.063\ /\ (0.5 \times 2.42) = 0.05m$

(5) Flow velocity of downward water vein in the water cushion ..... (See 3.4.5(3)4)(b))

$H_n = h_D + h_2 + V_2^2/2g = 0.15 + 0.24 + 0.52^2\ /\ (2 \times 9.8) = 0.40m$

$S = H_n\ /\ \sin\theta = 0.4\ /\sin 60.3° = 0.4\ /\ 0.869 = 0.46m$

$5.82d = 5.82 \times 0.05 = 0.29m$

So, $S = 0.46 > 5.82d = 0.29$

$\therefore V_{max} = 2.41V/\ \sqrt{(S/d)} = 2.41 \times 2.42\ /\ \sqrt{(0.46/0.05)} = 1.92m/s$

(6) Length of the water cushion (L_0) ..... (See 3.4.5(3)4)(c))

In the case of the width B_0 of the water cushion is equal to the width of downstream canal.

$L = X + \ell' = X + H_n\ /\ \tan\theta = 0.21 + 0.4/1.75 = 0.44m$

$L_0 \geq 2.5L = 2.5 \times 0.44 = 1.1m$

$\therefore L_0 = 1.5m$

(7) Confirmation calculation ..... (See 3.4.5(3)4)(c))

$P = \gamma_W \times B_0 \times H_n^2\ /\ 2 = 1 \times 0.5 \times 0.4^2\ /\ 2 = 0.04KN$

$3M = 3 \times \gamma_W \times Q \times V_{max}\ /\ g = 3 \times 1 \times 0.063 \times 1.92\ /\ 9.8 = 0.037$

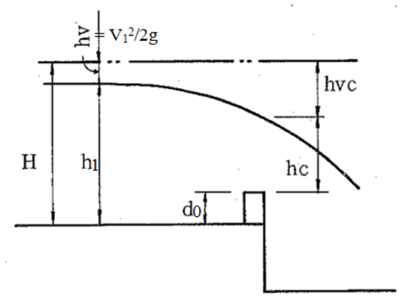
$\therefore\ P = 0.04 > 3M = 0.037$ OK

(8) Height of drop ($h_f + h_d$)

$h_f + h_d = y + H_n = 0.12 + 0.4 = 0.52m \approx 0.6m$

(9) Rising height of outfall (d_0)

$h_{vc} + h_c + d_0 = h_1 + V_1^2/2g$



$\therefore\ d_0 = h_1 + V_1^2/2g - h_{vc} - h_c = 0.24 + 0.52^2 / (2 \times 9.8) - 0.06 - 0.12 = 0.074$

$\approx 0.08\text{m}$

<Confirmation calculation>

$d_0 = 0.08 > 0.3 \times h_c = 0.3 \times 0.12 = 0.036$ OK

Refer to Excel format guide 4 for the calculation.

(10) Confirmation of the water level whether complete drop or incomplete drop..... (See 3.4.5(3)1)(c))

$$\left(\frac{h_2}{h_c}\right)^2 - \left(\frac{h_f}{h_c}\right)^2 = \frac{B_1}{B_2}\left\{3 - 2\left(\frac{B_1}{B_2}\right)\left(\frac{h_c}{h_2}\right)\right\}$$

$$\therefore 0 = \frac{B_1}{B_2}\left\{3 - 2\left(\frac{B_1}{B_2}\right)\left(\frac{h_c}{h_2}\right)\right\} - \left(\frac{h_2}{h_c}\right)^2 + \left(\frac{h_f}{h_c}\right)^2$$

The value of h_2 (the water level of the boundary area between an area with complete drop and an area with incomplete drop) is calculated by trial and error calculation. Refer to Excel format guide 4 for calculation of above formula.

The result of calculation, $h_2 = 0.49$m

Actual downstream canal water depth is 0.24m, so this water flow is complete drop.

Excel format guide 4. Drops

#Please input data into yellow cell

(1) Basic data for design

Design water discharge (Q)	0.063	m3/s		
Upstream canal width (B1)	0.5	m		
Upstream canal water depth (h1)	0.24	m		
Upstream canal water velocity (V1)	0.52	m/s		
Downstream canal width (B2)	0.5	m		
Downstream canal water depth (h2)	0.24	m		
Downstream canal water velocity (V2)	0.52	m/s		
Width of outfall (B)	0.5	m	← Use for design	※This value is assumption at first
Width of the water cushion (B_0)	0.7	m	← Use for design	※This value is assumption at first
Head of energy line to the center of water vain (Z)	0.30	m		※This value is assumption at first
Depth of water cushion (h_b)	0.15	m	← Use for design	※This value is assumption at first

(2) Upstream canal

Discharge per unit width (q)	0.13	m3/s		※In the case of q>2, formula has to be changed
Length of upstream canal (L)	1.58	m		
∴ For design, L =	1.60	m	← Use for design	

(3) Critical depth (hc), Specific energy at the critical depth of upstream canal (H), Head of energy line to the center of water vain (Z), Vertical distance measured downward from the outfall's downstream end (y)

Critical depth (h_c)	0.12	m
Water head of velocity in the case of Critical depth (h_{vc})	0.06	m
Specific energy at the critical depth of upstream canal (H)	0.18	m
Vertical distance measured downward from the outfall's downstream end (y)	0.12	m

(4) Shape of water vein
In the case of with afflux,

(a) Track of the center of water vein (X)	0.21	m	※In the case of without afflux, formula has to change
(b) Inclination angle of water vein (tanϴ)	1.75		
ϴ	60.3	°	
rad	1.1		
(c) Velocity at the center of water vein (V)	2.42	m/s	
(d) Thickness of water vein (d)	0.05	m	

(5) Flow velocity of downward water vein in the water cushion

Water depth at the end of the water chshion(H_n)	0.4	m
Penetration distance (S)	0.46	m
5.82d	0.29	m
S	>	5.82d

Flow velocity at an arbitrary point in the penetration direction which is on the extention of the center entry angle of falling water vein (V_{max})	1.92	m/s	※In the case of S≦5.82d, formula has to be changed

(6) Length of the water cushion

The horizontal distance which is from the outfall section of the falling water vein to the location at which the central portion of water vein comes in contact with the bottom of the water cushion (L)	0.44	m	
Length of the water cushion (L_0) >	1.1	m	※In the case of B_0=B_2, formula has to be changed
∴ For design, L_0 =	1.5		← Use for design

(7) Confirmation calculation

Hydrostatic pressure at the end of the water cushion (P)	0.04	KN	
Percentage (Force) of momentum change per unit time at a location at which a falling water vein reaches the bottom of the water cushion (M)	0.0123	KN	
∴ 3M =	0.037		
P	>	3M	
	OK		← When this cell is "NG", B_0 or h_D has to be change

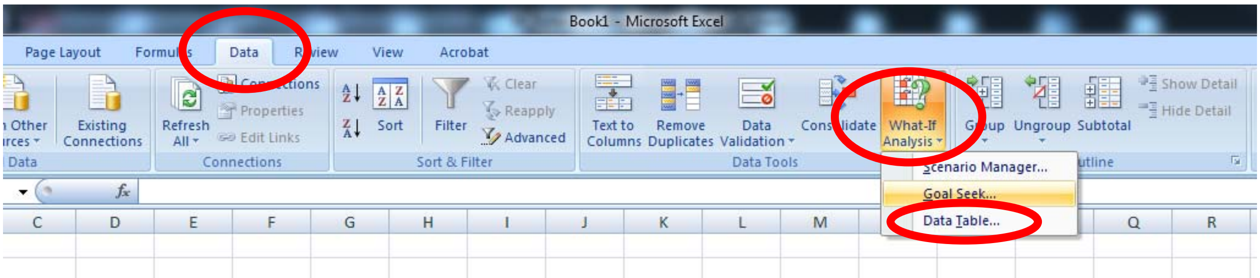
(8) Height of drop (h_f + h_D)	0.52	m	
∴ For design, h_f + h_D =	0.60	m	← Use for design
(9) Rising height of outfall (d_0)	0.074	m	
∴ For design, d_0 =	0.08	m	← Use for design
d_0	>	$0.3h_c$	
	OK		← When this cell is "NG", d_0 has to be changed

(10) Confirmation

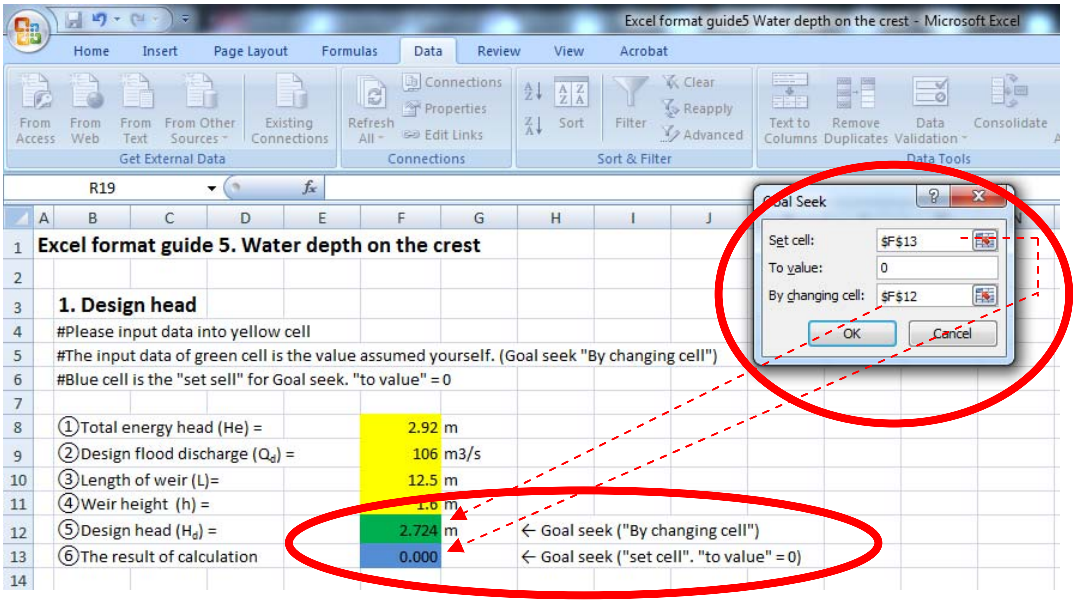
hf	0.45	m	
h2	0.49	m	←Goal seek "By changing cell" ※This value is assumption at first
	0.000		← Goal seek ("set sell". "to value" = 0)
Above h2	>	0.24	←Downstream canal water depth (h2)
	OK	= Complete drop	
			When this cell is "NG", Z or h_D has to be changed

5.3 How to use "Goal seek"

There is a function named "Goal Seek" on Excel.
This function can simplify trial and error calculation.
To use this function, it has to go to [Data] tab on Excel, then select [What-IF Analysis], and select [Goal Seek]. Place of item is shown below.



After select [Goal Seek], it can be selected "Set cell" which is having formula. Then enter a figure into "To value" which is target value.
Finally it can be selected "By changing cell" which is a cell to change figure to get target value. This cell's figure is an assumption value at first.
After those three cells are set on Goal Seek and click on OK, Excel calculates to find target value by itself and show that value under "By changing cell".
There is some guidance about "Set cell", "To value" and "By changing cell" in Excel format guide on this manual as follows.



Japan International Cooperation Agency (JICA) & Oromia Irrigation Development Authority (OIDA)
The Project for Capacity Building in Irrigation Development (CBID)

<Example>
In this case above,
“Set cell” is a ⑥ cell (blue color cell) which is having formula about the calculation of Design head,
“To value” is 0 because the formula about the calculation of Design head should be close to zero as much as possible,
“By changing cell” is ⑤ cell (green cell). This cell’s figure is an assumption value at first.
After those three cells are set on Goal Seek and click on OK, the value of “By changing cell” is shown most proper value (target value) to be close to zero as the answer of formula about the calculation of Design head automatically.

References

1. Engineering Manual for Irrigation & Drainage, Canal Works, The Japanese Institute of Irrigation and Drainage, 1987

2. Land Improvement Project Plan Design Standard, Design Canal Works Technical Document, The Japanese Institute of Irrigation and Drainage, 2002

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Manual for Runoff Analysis	Mr. Yasukazu Kobayashi	Runoff Analysis	LANDTEC JAPAN, Inc.
Manual of GIS for ArcGIS (Basic & Advanced Section)	Mr. Ron Nagai	GIS Application	KOKUSAI KOGYO CO., LTD.
Manual on Land Use Classification Analysis Using Remote Sensing	Mr. Kazutoshi Masuda	Remote Sensing	KOKUSAI KOGYO CO., LTD.
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Strengthening Irrigation Water Users Association (IWUA) Guideline	Mr. Yasushi Osato	Strengthening of WUA	Nippon Koei Co.
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<tr><th>Name of Guidelines and Manuals</th><th>Name</th><th>Field</th><th>Affiliation</th></tr>
<tr><td rowspan="2">• Guideline for Irrigation Master Plan Study Preparation on Surface Water Resources</td><td>Mr. Ermias Alemu Demissie</td><td>Irrigation Engineer</td><td>Lecturer in Arba Minch University</td></tr>
<tr><td>Mr. Zerihun Anbesa</td><td>Hydrologist</td><td>Lecturer in Arba Minch University</td></tr>
<tr><td rowspan="2">• Technical Guideline for Design of Headworks
• Technical Guideline for Irrigation Canal and Related Structures</td><td>Mr. Ermias Alemu Demissie</td><td>Irrigation Engineer</td><td>Lecturer in Arba Minch University</td></tr>
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<tr><td>• Construction Control Manual</td><td>Dr. Hiroaki Okada</td><td>Construction Supervision (Sokido/Saraweba SSIP)</td><td>Sanyu Consultants Inc.</td></tr>
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<tr><td>• Technical Guideline for Design of Headworks
• Construction Control Manual</td><td>Mr. Kenjiro Futagami</td><td>Facility Design/Construction Supervision</td><td>Ministry of Agriculture, Forestry and Fisheries</td></tr>
<tr><td>• All Guidelines and Manuals</td><td>Mr. Hiromu Uno</td><td>Chief Advisor/Irrigation Technology</td><td>Ministry of Agriculture, Forestry and Fisheries</td></tr>
<tr><td>• Manual for Runoff Analysis
• Manual of GIS for ArcGIS (Basic & Advanced Section)
• Manual on Land Use Classification Analysis Using Remote Sensing</td><td>Mr. Nobuhiko Suzuki</td><td>Water resources planning</td><td>Ministry of Agriculture, Forestry and Fisheries</td></tr>
<tr><td>• Guidance for Oromia Irrigation Development Project Implementation
• Study and Design Technical Guideline for Irrigation Projects
• Technical Guideline for Design of Headworks
• Technical Guideline for Small Scale Reservoir
• Construction Control Manual
• Guidance for Preparation of Operation and Maintenance Manual
• Irrigation Water Users Association Formation and Development Manual
• Strengthening Irrigation Water Users Association (IWUA) Guideline
• Small Scale Irrigation Water Management Guideline</td><td>Mr. Naoto Takano</td><td>Facility Design/ Construction Supervision</td><td>Ministry of Agriculture, Forestry and Fisheries</td></tr>
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