BANGLADESH WATER DEVELOPMENT BOARD
THE PEOPLE'S REPUBLIC OF BANGLADESH

THE PROJECT FOR CAPACITY DEVELOPMENT OF MANAGEMENT FOR SUSTAINABLE WATER RELATED INFRASTRUCTURE IN THE PEOPLE'S REPUBLIC OF BANGLADESH

FINAL REPORT
ANNEXES

SEPTEMBER 2017

JAPAN INTERNATIONAL COOPERATION AGENCY
(JICA)

IDEA Consultants, Inc.
INGEROSEC Corporation
EARTH SYSTEM SCIENCE Co., Ltd.

GE
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LIST OF ANNEXES

ANNEX 1:
Design Manual for River Embankment

BANGLADESH WATER DEVELOPMENT BOARD
THE PEOPLE'S REPUBLIC OF BANGLADESH

DESIGN MANUAL
FOR
RIVER EMBANKMENT IN BANGLADESH

September 2017

PREPARED BY
THE PROJECT FOR CAPACITY DEVELOPMENT OF MANAGEMENT
FOR SUSTAINABLE WATER RELATED INFRASTRUCTURE

Table of Content

Preface

The Standard Design Manual, comprising information and guidelines necessary for detailed design of structures pertaining to flood management works, irrigation works, drainage and road works of BWDB, was compiled by BWDB based on various overseas texts, publications, manuals and designs, etc. in 1995. The Design Manual for River Embankment in Bangladesh (Below, referred to as "design manual") is prepared after reviewing the concerned sections of the current Standard Design Manual that pertains to river embankment design while taking the following points into consideration.

(1) Almost the entire national land of Bangladesh is spread out over the downstream floodplains of three major rivers, the Ganges River, the Brahmaputra (the Jamuna River in Bangladesh) and the Meghna River; The topographical gradient is extremely gentle, and soils in tidal areas and Haor areas largely comprise silt and clay, whereas soils along major rivers comprise silt in surface layer, and sandy soil is assumed to accumulate under surface layer. Since embankments are constructed under such topographical, morphological and soil conditions, these factors should be taken into consideration when preparing the manual.

(2) After the construction of embankments and associated facilities, it is assumed that they cannot be completely free from damage as they are exposed to river flows and various other external environmental factors. Accordingly, it is important to conduct inspection and maintenance of the embankments and facilities after construction and people in charge of inspection and maintenance must properly understand the roles of embankments and associated facilities and the way of thinking of their design and construction. The detail explanations of fundamental idea behind design are described as much as possible in the manual.

(3) Because embankments are long, made of soil obtained from the vicinities and filled on the foundation ground, it is difficult to precisely specify the soil structure of each cross section of embankments and ground foundations. Furthermore they are exposed to change of water level and rainfall as external forces, it is difficult to strictly evaluate safety at each cross section of great length of embankments. On the other hand, in case that sufficient set back distance is secured or river bank protection is provided, embankments that have been constructed so far are considered to be safe with respect to normal infiltration of river water and rainfall at flood time, because embankments have been constructed in consideration of knowledge based on long construction histories in foreign countries and a recent flood damage in Bangladesh. Accordingly, embankments are designed based on prescribed shape method based on past experience, also in consideration of characteristics of ground foundations. Verification of embankment safety by means of theoretical engineering design methods will be conducted as needed. The current Standard Design will be supplemented with requisite clarification on the above thinking.

(4) The Design Manual for River Embankment is compiled with respect to design methods for securing safety of embankments. Embankment safety with respect to erosion which is main factor of embankment failure, should be basically secured by providing the sufficient set-back, however, in cases of meandering rivers where embankments are close to the riverbank and the set- back cannot be provided, it is necessary to provide countermeasures against local scouring integrally with embankments.

The first guidelines for design of river bank protection were prepared by BWDB in 1995, while the latest Guidelines for River Bank Protection were prepared in 2010. The Guidelines for River Bank Protection cover all erosion countermeasures including not only design of counter measures in terms of river bank erosion but also basic knowledge on morphology of river bed, recent measures to protect river embankment and foot of embankment against local scouring. Therefore, local scouring countermeasures necessary for embankment design should be designed in reference to the "the Guidelines" and reference parts are specified in the design manual for river embankment.

(5) In conventional river embankment design, not a lot of attention have been given to earthquake countermeasures for the following reasons: 1) embankments generally are only 3 or 4 meters in height; 2) there is little likelihood that earthquake and flooding occur simultaneously. Even if embankments are damaged by earthquake, because they are earthen levees, the damaged embankment could be restored before next flooding or high tide. Greater priority is placed on protection against flooding than earthquake countermeasure since earthquake takes place rarely. Accordingly, it has been generally thought that it is not necessary to consider the anti-earthquake measures in Bangladesh at present.

However, since high water level exists longer period in flood season and there is an increasing human life and property along rivers in accordance with the rapid increase of population in Bangladesh, the introduction of verification of river embankment safety with respect to Earthquake in Japan into Bangladesh is useful for assessment of the flood damage after the earthquake.

(6) This manual is prepared for Design of River embankments. Therefore, manual for design of Sea dykes is not included, but described as a reference.

(7) Concerning soil tests, only a few limited agencies have the capability to implement sophisticated soil tests. Accordingly, in preparing the design manual, design procedures based on the result of sophisticated soil tests are avoided as much as possible. But the verification based on the basic soil test is incorporated in the new manual.

(8) Two annexes and one appendix are attached to this manual. The first annex is "The PILOT project Design Report", and the second annex is "The Assessment Report on Earthquake

Resistance of the Embankment”. The appendix is “The assessment on Stability of existing embankment which is subject to flood water”.

The PILOT project Design Report is the embankment design specification of the Manu river in Moulvibazar, this embankment was actually designed and constructed based on this manual.

The Assessment Report on Earthquake Resistance of the Embankment specify the method for assessing slope stability of the embankment at Bogra in Bangladesh during an earthquake in accordance with this Design Manual as a reference.

These two annexes are expected to be helpful for designer of river embankment in Bangladesh.

The Assessment Report on Stability of existing embankment which is subject to flood water roughly shows the stability and seepage safety of existing embankments in Bangladesh. The embankments in Bangladesh are roughly classified to 4 typical embankment categories. The assessments were conducted for each embankment of 4 typical categories to identify the factors of embankment failures.

1. Prerequisites concerning River Embankment Design

When designing river embankments, it will be done based on the following characteristics and constraints, etc.

(1) Geographical and Topographical Characteristics

Bangladesh shares borders with India in the west, north and east, and with Myanmar in the southeast, while it faces onto the Bay of Bengal in the south. The national land spreads over the downstream flood plains of three giant rivers, the Ganges, the Brahmaputra, and the Meghna. Bangladesh, which is situated in the lowest reaches of the Ganges and the Brahmaputra rivers, accounts for roughly 5% of the Ganges watershed and 6% of the Brahmaputra watershed. As for the Meghna River, Bangladesh accounts for more than 40% of the entire watershed area.

As per land contour of Bangladesh obtained from 1963 irrigation planning map and map of subsequent surveys, roughly half of the national land of Bangladesh is low land at an altitude of not higher than 7 meters, where the Ganges and the Brahmaputra rivers are at an altitude of 30~40 meters on the border with India. The Brahmaputra River runs down from this altitude to the Bay of Bengal over a river course of approximately 400 kilometers. Accordingly, the river course gradient is extremely gentle at not more than 1/10,000 and the flood continuation time is very long.

Meanwhile, the upper reaches in the north of the Meghna River passes through the Meghalaya Mountains, the rainiest place in the world, while the middle reaches are surrounded by extremely low-lying land. Here, 10-meter contour lines penetrate deep into upper reach of the Meghna watershed so that an extremely deep basin is formed during flooding.

In the southwest of Bangladesh, many branch rivers of the Ganges and the Padma river flow between polders, and polder embankments are constructed to surround the areas to protect them from a flood. The river embankment in tidal area that are influenced by the ebbs and floods of the Bay of Bengal, river bank erosion is caused by the tidal movements.

Moreover, because almost all the rivers in Bangladesh have their upper reaches in other countries such as India, it is not easy to obtain information and data on embankment facilities, hydrological conditions, and river conditions in the upstream areas. Accordingly, when deciding the design high water level and height of embankments, decisions are generally based on river hydraulic and hydrological data within Bangladesh.

(2) Soil Characteristics

Bangladesh is situated on the downstream floodplains of three major rivers. Sediment carried down from the upper reaches is sorted as fine particle sediment and forms ground comprising mainly silt and sandy silt over the entire country. Accordingly, the river banks are vulnerable to erosion, and riverbank erosions occur in various places of rivers during the flood season and also at other time like receding flood, change of course of river etc.

Moreover, the grounds in Haor area where a flood forms deep water basin in flood season and in tidal area of the bay of Bengal which is influenced by ebbs and flow comprise soil with high distribution ratio of clay, and ratios of sand, silt, and clay that comprise the fine particle sediment differ greatly making different types of ground according to river characteristics and topographical characteristics. Accordingly, the composition of soil that can be used as embankment materials greatly varies regionally.

(3) River Characteristics

Major rivers with large river basins, have deep water depths, wide river widths and gentle gradients. Such characteristics form braided rivers, and large-scale riverbank erosion is caused by the movement, growth and disappearance of sand bars. Moreover, flooding continues for a long time and hundreds of meters of riverbank can be eroded during a single flood season, and there is also concern over piping and circular sliding of flood embankments.

Since tributaries of the Meghna River and other numerous small and medium tributaries in the north and east of Bangladesh have their upper reaches in a fold mountain areas of India, rainfall in the upper reaches tend to suddenly rush down and cause flooding in Bangladesh. Moreover, because the rivers meander with gentle gradients after they enter Bangladesh, water levels rise suddenly, causing flash floods that sometimes flow over bank level and cause inundation. In addition, local scouring is triggered on the outer banks of river bends, sometimes causing embankments failure.

Because the Meghna River, which contains numerous flash flood tributaries such described above, flows through an extremely low altitude basin (Haor) in the northeast of Bangladesh, its waters become very deep and submerge river embankments during the rainy season, meaning that the embankments protect against flooding only during the dry season till May. Accordingly, the river water overflows embankments during the pre-monsoon season when the dry season changes to the rainy season.

(4) Plan form of Flood Protection works

In Bangladesh, polder embankments (encircled levees) are frequently adopted in order to protect target areas (farmland, houses, etc.), while the approach of confining floodwaters inside the river course through building embankments on both banks is not very common. Moreover, because embankments in upstream areas are repeatedly breached as a result of bank erosion, flooding tends to occur in the upper reaches and accordingly water level in the lower reaches is not too high.

In the case of Major rivers, embankment heights tend to be not more than 4 or 5 meters; water pressure during flooding is low, and the external force applied to embankment bodies is small. As such, the effects of embankment damage by earthquake are not also large.

(5) Financial Constraints

In Bangladesh, because of financial constraint and on the other hand, due to available abundant manpower, embankments are largely constructed by manual labor. Thus, it is necessary to adopt construction management and quality control methods suited to this type of construction.

Moreover, when constructing new retired embankments on large rivers, as large quantities of embankment soil are needed, embankments are often constructed using sand that is dredged from sandy riverbeds. On embankment crests and slopes, surface sealing by clay is carried out in order to counter erosion caused by rain and water flows. Because clay becomes degraded due to sunlight and rainfall, it is necessary to implement maintenance work regularly.

Now a days also compacted embankments with machine are being constructed but that involves higher cost.

2. Basics of Embankment Design

2.1 Required Functions of Embankments (General)

> Embankments shall be designed to be safe with respect to 1) the ordinary actions (scouring and seepage) of flowing water up to the design flood water level (the design high tide level in high tide areas), and 2) deformation of foundation ground.
>
> River embankments in Bangladesh are of different shape & size depending on geographical location as for soil of foundation ground, material to be used for embankment, duration of high water level, types of embankment failure and so on. Design of river embankment should be performed in due consideration for such characteristics.
>
> With respect to seismic action, at times of normal water level, seismic resistance shall be evaluated and relevant necessary countermeasures shall be taken so that secondary damage will not be caused by flooding even if earthquakes cause embankments failure.

(Detail explanation)

1. General

Embankments as a rule are constructed by adjacent soil. Although earthen embankments have numerous advantages such as 1) construction costs are generally low, 2) they are resistant to degradation, 3) it is easy to conduct heightening, widening and repair works, 4) they can be easily integrated with foundation ground. On the other hand, since potential causes of embankment failure are piping by seepage from the embankment body or foundation ground, scouring caused by flowing water, and sliding by infiltration of river water and rainfall~~,~~ it is necessary for embankments to have sufficient safety against these risks.

Moreover, concerning embankments on soft ground, it is necessary to conduct verification with respect to sliding not only in the embankment body, but also in the foundation soil, and subsidence of the foundation ground. Because it is assumed that breaches could be caused by overflow due to sliding of embankment and lowering of the embankment crest level.

There is little possibility of earthquake and flooding occurring simultaneously, and even if embankments are damaged by earthquake, restoration is relatively easy and the minimum embankment functions can still be secured because they are earthen embankment, and therefore their restoration could be finished before occurrence of coming flooding or high tide. Therefore, greater priority is placed on protection against flooding, which occurs more frequently than earthquakes, and no counter measure against earthquake has been taken before.

However, strengthening method used in Japan is introduced in 4.4.6 in this manual, because there are some sections where high river water level continues for long time in Bangladesh and it might become necessary to consider flood damage during the period from occurrence of earthquake to restoration of embankments.

Investigation of embankment damage caused by earthquake in Japan reveals that the extent of damage is largely determined by the strength of foundation ground. In particular, damage tends to be extremely significant when the foundation ground becomes liquefied. Even in that worst case, there was no embankment ending up entire collapse and at least 25% or more of embankment height remained.

However, there are possibility that secondary disaster (inundation disaster) by overflow of river water could be caused depending on the degree of embankment subsidence in the place where land height is lower than river water level. Accordingly, countermeasures are taken into consideration for such cases.

2. Functions required for embankments

(1) Embankments along major rivers (the Jamuna, the Ganges and the upper Meghna)

These rivers have an extremely gentle gradient of 1/10,000 or less after they enter Bangladesh. At times of flooding, the high water level continues for a long time and embankments are subject to seepage. Meanwhile, despite the gentle river gradients, because of deep depth of river, flow velocity at flood times reaches up to around 2.0 meters/second.

On embankments themselves, there is concern over breaching caused by piping at the toe of embankment built of sandy soil on the land side due to seepage flows, while there is a possibility that residual water inside embankments will trigger slope slide on river side after the water level drops. Generally, embankments are constructed of clayey silt in Bangladesh. However, since a large amount of soil is needed to construct embankments along the Major rivers, sometime the embankments are constructed with sand that is dredged from riverbeds, and embankment crests and slopes are covered by clay to prevent from erosion by river flow, infiltration of river water and rainfall. If surface clay is not maintained adequately after construction, it is sometimes abraded by rainfall and sunshine, and over the long term this causes embankments to become prone to erosion caused by water flows. Therefore, it is important for surface clay to be covered with turf and conducted maintenance work on embankments after construction.

Moreover, because there are many cases that the foundation ground of embankments on the Major rivers are composed of sandy silt with high permeability, even if surface covering by clay is provided on embankments, there is still concern over the piping and failure of embankments resulting from long-term seepage through the foundation ground. Accordingly, depending on the situation, it may be necessary to implement seepage prevention works.

(2) Embankments in tidal areas

In tidal areas, Branch Rivers originating mostly from the Ganges, flow between large polders in the form of net and into the Bay of Bengal. These branch rivers have extremely gentle gradients, so the effects of oceanographic phenomena (high and low tides, waves, cyclones) in the Bay of Bengal is likely to extend far upstream reaches.

The height of embankment crest need to have some free board in consideration of the effects of tides and waves and storm surge, moreover, it is necessary for embankment crests and slopes influenced of erosion by waves to be covered by hard material in order to protect against waves.

Because ground in this area predominantly consists of clay and embankment bodies usually comprise clayey silt, compaction cannot be adequately performed and many small openings remain in construction. However, it is unlikely that embankment failure caused by piping triggered by seepage flows occurs.

At the outer banks side of river embankments (polder embankments) on bends of Branch Rivers, bank erosion is caused by ebbs and floods at higher water levels during flooding and this bank erosion can be seen numerously. However, because the local ground comprises clayey silt, the speed of bank erosion is slow.

Meanwhile, in the areas that are enclosed by embankments, rice farming and even inland water fisheries like shrimp culture are carried out, and sometimes the river embankments (polder embankments) are subject to public cutting in order to secure water for such activities. Accordingly, from the viewpoint of protecting embankments, it is necessary to provide facilities that can supply water to the inland areas properly.

(3) Embankments in Haor area

In the north and east of Bangladesh, large quantities of floodwater flow into Haor area through numerous tributaries from Meghalaya State (the place that has the most rainfall in the world) and the Assam region of India during the flood season and form the Upper Meghna River that flows from east to west in Haor area.

The watershed of the Upper Meghna River forms an extremely low altitude basin that is filled with flood water during the flood season.

River embankments in Haor play a role to flow down flood to prevent from inundation during the dry season, however, embankments are completely submerged under water during the flood season (water level reaches 2~3 meters higher than the crest level of embankment).

In the pre-monsoon period (April-May) that marks the transition from the dry season to the rainy season, river water levels rise and river water overflows embankments. To ensure that the embankments would not be breached, river water is allowed to enter inside the polders via regulators so that the difference of water level inside and outside of the polders is reduced. Embankments have a certain degree of strength against overflow, because materials of embankments are composed of adhesive clayey silt. However, it is desired and tried to delay the timing of drawing of river water into the polder for their farming.

Due to this fact, overflow often takes place when the water level difference between inside and outside of the polder is still large. Under this situation, if overflow takes place, scouring of embankment toe on the land side becomes more serious. In addition to that, “Public cut” of embankment has been often conducted for the water traffic convenience.

Compaction of embankment in Haor is conducted either mechanically or by manual laborers using rammers. When compaction is performed by manual laborers, it is difficult to achieve sufficient compaction because embankment is made of clayey silt. Accordingly many openings still remains in embankments after construction. However, piping is unlikely to occur due to characteristics of embankment materials.

(4) Embankments in areas affected by flash floods

Many of the small and medium-size rivers that originate in India and flow into the Upper Meghna and its tributaries have small watersheds and narrow river widths; moreover, there is hardly any set-back distance on the outer banks of river bends. The upper basins of such rivers are situated in precipitous mountains (The altitudes of some mountains are higher than 1,000 meters above mean sea level), and the altitude of rivers suddenly become very low after those rivers enter into Bangladesh and become meandering rivers with very gentle longitudinal gradient. The water level of rivers flowing over the flatlands in Bangladesh suddenly rises at time of flood, often leading to flood inundation in the lower reaches. Moreover, local scouring occurs on the outer banks of numerous river bends, and combined with the fact that there is hardly any set-back distance in such areas, local scouring occurs at toe of embankment on river side and directly leads to the collapse of entire embankments.

Accordingly, it is necessary to provide foot protection works as a counter measure for local scouring on the outer banks of bends; furthermore, in order to address increased flow velocity and secondary flows caused by bends, it is desirable to cover embankment slopes by hard materials as blocks with projection. However, since the blocks themselves can also be subject to the force of flowing water, it is necessary to consider the stabilization of blocks.

2.2 Types of Embankments

Embankments may be classified into following three major types depending on the nature of protection they provide and their locations.

1) Full flood protection embankments
2) Submersible embankments
3) Sea dikes

(Detail explanation)

1. Full flood protection embankment is aimed at preventing the highest flood from flowing across the embankment throughout a year. The design flood level is determined from the view of return period of flood depending upon the degree of importance of the inside of embankment.
   Embankments in the areas affected by flash flood and embankments in tidal areas mentioned above are included in this type.

2. Submersible embankments are usually built to provide protection during dry season and pre-monsoon season; therefore they are overtopped and remain submerged during the monsoon.

3. Sea dykes are usually built along the coastal belt to protect lands against flooding and intrusion of saline water, in addition, these embankments must give protection against tidal surge during cyclones. Sea dyke, distinguished from river embankment, are constructed with concrete covering over crest and both slopes in consideration of overtopping waves. Moreover, in view of risk of overtopping waves, parapet walls and water collecting channels are provided according to necessity.

2.3 Embankment Design Procedure

River embankment design should be basically conducted based on the prescribed shape method as a first step, and verification of design should be conducted to ensure the following safety as needed:

1) Embankment safety with respect to seepage
2) Embankment safety with respect to erosion, sliding and piping
3) Embankment safety with respect to earthquake
4) Embankment safety with respect to bearing capacity of foundation ground

(Detail explanation)

1. General

River embankments are long structures. They are made of soil with different ingredients and have uneven strength depending on the location of construction. Since it is difficult to examine the safety of embankment with each short interval, they have conventionally been designed based on the prescribed shape method which specifies minimum required levels of cross-sectional shapes (crest width, etc.).

River embankments should be designed to prevent flood from inundating, therefore they should be stable for seepage and erosion caused by flood. Further, in case that river embankments are constructed on soft ground, it is assumed that subsidence of foundation ground lead to subsidence of embankments is caused by construction of embankments, with the result that overtopping would bring about.

Stability of river embankments for earthquake has not been taken into consideration so far. But it could be necessary to consider the damage caused by embankment collapse triggered by earthquake and to take necessary measures.

Verification of embankment safety with respect to seepage, erosion in flood time, and earthquake and bearing capacity on soft ground in non-flood time should be conducted at the locations identified as needed by the survey.

River embankments should be safe not only from circular sliding and piping caused by infiltration of rainfall and river water but also from sudden draw down of river water level after the flood for all embankment sections.

River embankments should be safe from erosion caused by running flow of river, is also required at all embankment sections.

Meanwhile, subsidence of embankment is caused by liquefaction of ground due to the earthquake, is also required at such locations where inundation may take place in due to the embankment subsidence in low-lying areas at the non-flood time, on the premise that both earthquake and flood are unlikely to take place spontaneously. .

In case that bearing capacity of foundation ground is not sufficient because of soft soil and that set back distance is not sufficient, it is possible that circular sliding with large radius through foundation ground occurs even when river water level is low in dry season.

2. Flow of Design procedure

The basic steps of embankment design are described below.

(1) Collection of basic hydrological and hydraulic data

(2) Basic hydrological and hydraulic analysis

(3) Identification of reaches where hydrological and hydraulic conditions are the same

(4) Basic investigation on crest level, crest width, set back distance etc. for design

(5) Basic design of cross sectional profile

(6) Identification of locations for verification of embankment

(7) Collection of data at the locations for verification

(Soil data of foundation ground and embankment etc.)

(8) Analysis on seepage (sliding and piping) and subsidence of foundation ground on soft ground.

(9) Verification of embankment safety for seepage, erosion, bearing capacity of foundation ground and prediction of inundation area in earthquake.

(10) Adjustment of cross section and design of necessary facilities like revetment

2.4 Alignment of Embankment

Embankments are constructed to protect human life, properties and farmland from flooding, and they must be maintained up to the planned required height. In the case of embankments on both banks, sufficient width between embankments must be provided in cross section composed by the low-water channel and set-back areas to pass the design flood discharge.
Determination of the alignment of an embankment is governed by technical, economical and morphological and social considerations.

(Detail explanation)

The embankment alignment is set up to achieve the required functions of the embankment with also giving consideration following issues for securing safety of the embankment itself.

1) The required set-back should be secured in consideration of bank erosion. If impossible, protection work against erosion should be provided.
2) Peat soil should be avoided. If this is difficult to do so, either peat soil should be removed or the amount of subsidence of peat soil caused by embankment construction should be added to design flood height of embankment (extra banking).
3) Preferably alignment should avoid the land consisted of fair portion of clay, because there is possibility of lack of ground bearing capacity.
4) Land consisted of sand, where there is possibility of liquefaction of foundation ground, should be avoided as much as possible.
5) The embankment alignment should be as smooth as possible without having any sharp bends.
6) The embankment alignment should be fixed along high ground, while depressions should be avoided as much as possible.

Moreover, the following points should also be taken into account from the social and economic viewpoints.

1) The embankment alignment should avoid the areas that comprise soil that can be used as embankment materials.
2) The embankment alignment should generally not block current transportation routes.

2.5 Set-Back

Set back is the space between actual river bank and river side toe of the embankment.
Securing of set-back width is very important for safety of embankment, because most of embankment failure takes place due to bank erosion. Therefore, sufficient set-back width to protect embankments from bank erosion and scouring, must be provided between river banks and toe of embankment.
Width of set-back should be estimated based on the past erosion rate and morphological characteristics of the river.

(Detail explanation)

The width of set-back should be determined to protect embankments from bank erosion in considering the followings.

1) In places where bank erosion has repeatedly occurred in the past, the width of set-back should be specified based on the extent of past bank erosion. The width equivalent to 5-10 years of the current erosion should be provided.
   However, many flash flood rivers opt to meander and there are many local scours at outer banks of bends. There are many cases that securing sufficient set-back distance is difficult.
2) In places where embankments are constructed on both sides of rivers, sufficient width of set-back (floodway) must be secured in consideration of the design flood discharge
3) If sufficient width of set-back could not be provided, river bank protection works should be provided as needed.

2.6 Design Crest Level

Design crest level is determined by adding free board over the design flood level. The design crest level is composed of design flood level, height of wind set up (H_W) and height of wave run-up (Hz) and safety margin. .

In an actual construction, crest level should be determined including some extra banking in consideration of compressive subsidence of the embankment body, consolidation subsidence of foundation ground and crest damage caused by weathering.

In addition, a cross-sectional gradient of embankment crest should be around 5% for drainage purposes on the crest.

(Detail explanation)

1. Since embankments basically consist of earth, they are extremely vulnerable to overflow, embankments should be designed so as not to permit overflow of water at or below the design flood level, and it is also necessary to give some free board against temporary rises of water level

caused by flood waves, swell, splash and the like. Moreover, sufficient free board should be added to the design flood level to ensure safety for patrols and to deal with flowing objects such as driftwoods and others during floods when determining the embankment crest level.

2. Water level rise due to anticipated river-bed fluctuations, water level rise at bend sections and water level rise due to hydraulic calculation error, etc. should be taken into consideration for determining the design flood level.

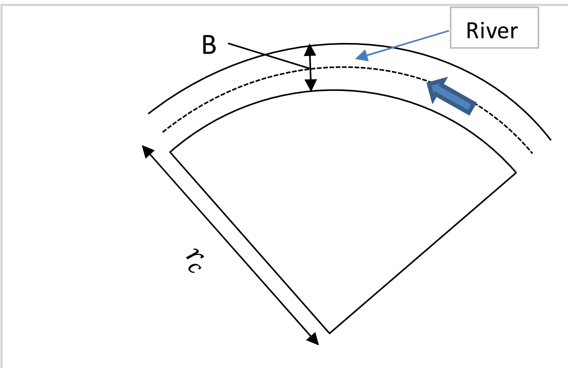
In addition, height of embankment is the difference between the design crest level of the embankment and the average ground level on which the embankment is constructed.

3. Water level rising occurs on the outer bank side of bend, on the other hand, water level drop occurs on the opposite side. The water level obtained by non-uniform flow calculation is that at center line of river. Water level rising (Δh/2) should be added to this for design flood level.

On this, it is not necessary to take water level rising into consideration in the case that r_c/B is more than 10.

Here, $$\Delta \mathrm{h} = \frac{B \cdot v^2}{g \cdot r_c}$$

Where; Δh; rising height raised by bend (m)
B; averaged width of both banks at bend section (m)
v; averaged current velocity (m/s)
r_c; curvature radius (m)



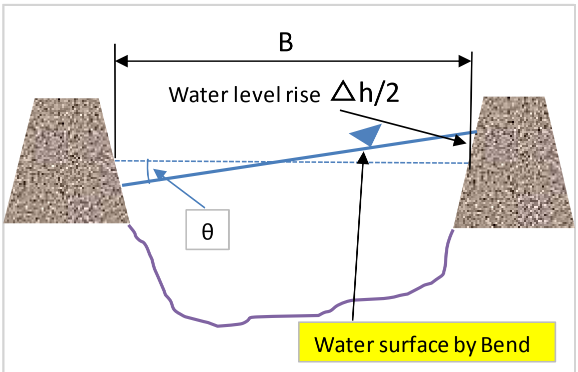


Fig-1 Plane view at Bend

Fig -2 Cross Section at Bend

(Reference)

When considering the balance of power of the unit volume (m) of water acting by gravity and by centrifugal force, it holds the following equation.

$$\mathrm{mg} \cdot \sin\theta = \frac{mv^2}{r_c}\cos\theta$$

Where θ ; Inclination of water surface

m ; Unit mass of water

$\Delta h / B = \tan\theta$

$$\therefore \Delta h = \frac{Bv^2}{gr_c}$$

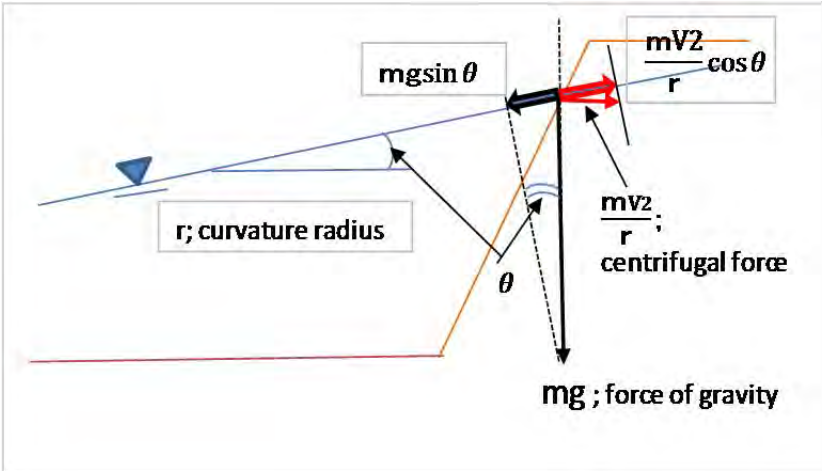


Fig-3 Equilibrium of force at bend

2.6.1 Design Flood Level

Design flood level should be different according to the types of embankments.

In case of full flood protection embankment, Design flood level should be determined depending on the following two cases;

i) Case where embankment is constructed only on a single side of the river
ii) Case where embankments are constructed on both sides of the river

(Detail explanation)

The design flood level should be calculated and specified according to the type of embankment. In case of full flood protection embankment, the level depends on whether embankment is constructed only on a single side of the river or on both sides of the river.

1. Full flood protection embankment

(1) Case where embankment is constructed only on one bank of the river

- Embankment needs to be constructed on one bank only, the design flood levels may be computed by frequency analysis of available annual maximum river level data of adjacent water level observatory.

(2) Case where embankments are constructed on both sides of the river

- In the case of river planning that flood water which has flowed out from mountains is confined in the river without flooding, embankments are constructed on both sides of the river on the plains in order to confine flood waters in the river.
- Water level records from the local water level station are those before the embankment is constructed. It is not logical to conduct frequency analysis based on such data because the data could be obtained under the situation that inundation occurred in upstream areas.
- The design flood level should be calculated with the assumption that embankments are constructed on both banks and there is no inundation in upstream areas.
- The design crest level is calculated as follows in the case of both sides of the river.

1) Selection of design flood scale intensity

2) Based on historical rainfall record, the occurrence probability corresponding to design rainfall (amount of rainfall, space-time distribution of rainfall) is calculated.

3) Design flood discharge corresponding to design rainfall is calculated by simulation (run-off model).

4) River water levels at each points corresponding to design flood discharge are calculated by simulation (non-uniform flow and so on.)

5) Design crest level is obtained by adding free board to design flood level.

2. Submersible embankment

Submersible embankments are constructed to provide protection against flooding during the dry season and pre-monsoon season (April and May), they become submerged during the flood season. Accordingly, in Haor areas where submersible embankments are constructed, the design flood level should be computed by frequency analysis of maximum river level data before some specified time during the year. e.g. 31 May or it may be 15th May depending upon local cropping pattern and harvest time.

In cases where regulators are constructed between submersible embankments, in order to prevent the connected submersible embankments from being washed away, the crest level of the submersible embankment should be aligned with the level of the regulator deck in 50-meter extent of embankments on either side of the regulators.

3. Sea dyke

Sea Dyke is designed to be distinguished from river embankment. Design tidal level can be obtained by adding tidal level deviation to Average Highest Tide Water Level which is the additional water level raised by the assumed largest size cyclone in general.

Review on crest level of Polder embankments, which have been constructed since1960, have been carried out taking into account rising of tidal level and influence of waves caused by Global Warming under the support of World Bank. [Coastal Embankment Improvement Project (CEIP-1) September 2012]

Design tidal level have been examined in CEIP-1. Design tidal level of each embankment are obtained by simulating water levels observed at 105 observatories in 38 cyclones landed on Bangladesh in consideration of 25 years occurrence probability scale.

The influence by Global Warming is estimated as 0.5m rising of sea level and 10% increasing of wind velocity based on the values for the time up to 2050 predicted by IPCC.

Further, crest height of Sea dyke is the height after adding the subsidence and freeboard (overtopping amount is restricted less than 0.5ℓ/s/m) to design tidal level.

2.6.2 Selection of Design Flood Occurrence Frequency

In case of full flood protection embankment, the design flood occurrence frequency, which is used for determining the embankment design height, should be adopted based on the importance of the target area in consideration of the followings

1) Areas where damage to agriculture is predominant: Flooding with occurrence frequency of once in 20 years
2) Areas where damage to human life, properties and facilities are predominant: Flooding with occurrence frequency of once in 100 years.
(Generally, embankments of the Jamuna River, the Ganges River and the Meghna River are of this type).

In case of submersible embankment, the design flood occurrence frequency is adopted based on the pre-monsoon flooding with occurrence frequency of once in 10 years

(Detail explanation)

1. The design flood level with respect to occurrence frequency (exceedance probability of hydrological value) should be estimated with use of a probability distribution model.

   1) The case of full flood protection embankment; annual peak value.
   2) The case of Submersible embankment; peak value in pre-monsoon season before 31st May, but it may be 15^{th} May depending upon local cropping pattern and harvesting time.

2. Since the observation data used in flood control planning is those of 20 or 30 years at most, it is necessary to extrapolate from collected data in order to estimate probable hydrological values with a return period of 50 or 100 years.

3. Some of temporary observatories have water levels recorded in only 10 years. Therefore, it should be noted that such data is too small to be adopted for calculation..

4. It should be noted that recorded water level data at the observatories facing broad water body have already included influence of wind set up.

〔Reference〕 Analysis of flood level occurrence frequency

Analysis of flood level occurrence frequency should be conducted upon confirming the steadiness of the collected hydrological data (annual peak values). Since not-steady data contains trends (long-term trend changes) and jumps (sudden changes), steadiness should be confirmed by means of the following procedure.

(1) Confirmation of steadiness

Steadiness means there is no unusual particular components other than the random fluctuations over time, and this should be reviewed as follows.

1) Review of periodicity of the hydrological data
2) Review of jumps in the hydrological data
3) Review of trends

(2) Frequency analysis of steady hydrological values

If the above reviews indicate that data comprises steady hydrological values, frequency analysis should be conducted according to the following procedure.

1) Listing of the candidate probability distribution models
2) Estimation of parameters of the probability distribution models
3) Evaluation of goodness of fit of candidate probability distribution models
4) Bias correction and stability evaluation of probable hydrological values
5) Determination of the probability distribution model

(3) Listing of candidate probability distributions

The following candidate probability distributions can be considered. The some examples of occurrence probability distribution in the case of annual highest water levels of the Manu River are shown in Fig 4.

1) Gumbel distribution (Gumbel)

$$F(x) = \exp\left[-exp\left\{-\frac{x-\xi}{\alpha}\right\}\right]$$

2) General extreme value distribution (Gev)

$$F(x) = \exp\left[-\left(1-k\frac{x-\xi}{\alpha}\right)^{1/k}\right]$$

3) Exponential distribution (Exp)

$$F(x) = 1-\exp\left\{-\frac{x-\xi}{\alpha}\right\}$$

4) Normal distribution

$$F(x) = \frac{1}{\sqrt{2\pi\sigma}} exp\left\{-\frac{1}{2}\left(\frac{x-\mu}{\alpha}\right)^2\right\}$$

5) Pearson III (LP3Rs, LogP3)

$\gamma > 0,\ \xi \leq x < \infty$; $F(x) = G\left(\alpha, \frac{x-\xi}{\beta}\right)/\Gamma(\alpha)$

$\gamma < 0, -\infty < x < \xi$; $F(x) = 1-G\left(\alpha, \frac{\xi-x}{\beta}\right)/\Gamma(\alpha)$

6) General Pareto distribution (non-annual value distribution) (GpExp)

$$F(x) = 1-\left(1-k\frac{x-\xi}{\alpha}\right)^{1/k}$$

Where, $F(x)$; probability distribution function

ξ; Location parameter

α; Scale parameter

k; Configuration parameter

$G(\alpha, z) = \int_0^z t^{\alpha-1}\exp(-t)\,dt$

(4) Estimation of parameters of the probability distribution models

Parameters estimation should be conducted by the following methods:

1) Moment method
2) L moment method
3) Maximum-likelihood method

(5) Evaluation of goodness of fit of candidate probability distribution models

The SLSC (Standard Least Square Criteria) method should be used to evaluate the goodness of fit of the candidate model to the hydrological data.

$$SLSC = \frac{\sqrt{\xi^2}}{|S_{99}-S_{01}|}, \quad \xi^2 = \frac{1}{n}\sum_{i=1}^{n}(S_i - S_i^*)^2$$

Where, $S_{99} - S_{01}$; normal variate of probabilistic distribution for non-exceedance probability of 0.99 and 0.0

n; sample size

S_i; Normal variate transformed from order statistics by estimation parameter

S_i^*; Normal variate transformed from quantile corresponding to plotting position that obtained from probabilistic distribution model by estimation parameter

The goodness of fit is deemed sufficient if the SLSC is not greater than 0.04. In addition, goodness of fit of the model is also ascertained through plotting on probability paper in addition to performing the evaluation based on SLSC.

Here, the Cunnane plot, which is compatible with numerous distributions, is used as the plotting position. Here, the exceedance probability P_i of the i'th value of the hydrological data ($X_1 \geq X_2 \geq X_3 \geq \cdot\cdot\cdot \geq X_n$) is expressed as follows:

$$P_i = (i-\alpha) / (n-2\alpha+1)$$

Where, n: Sample size, α: =0.4

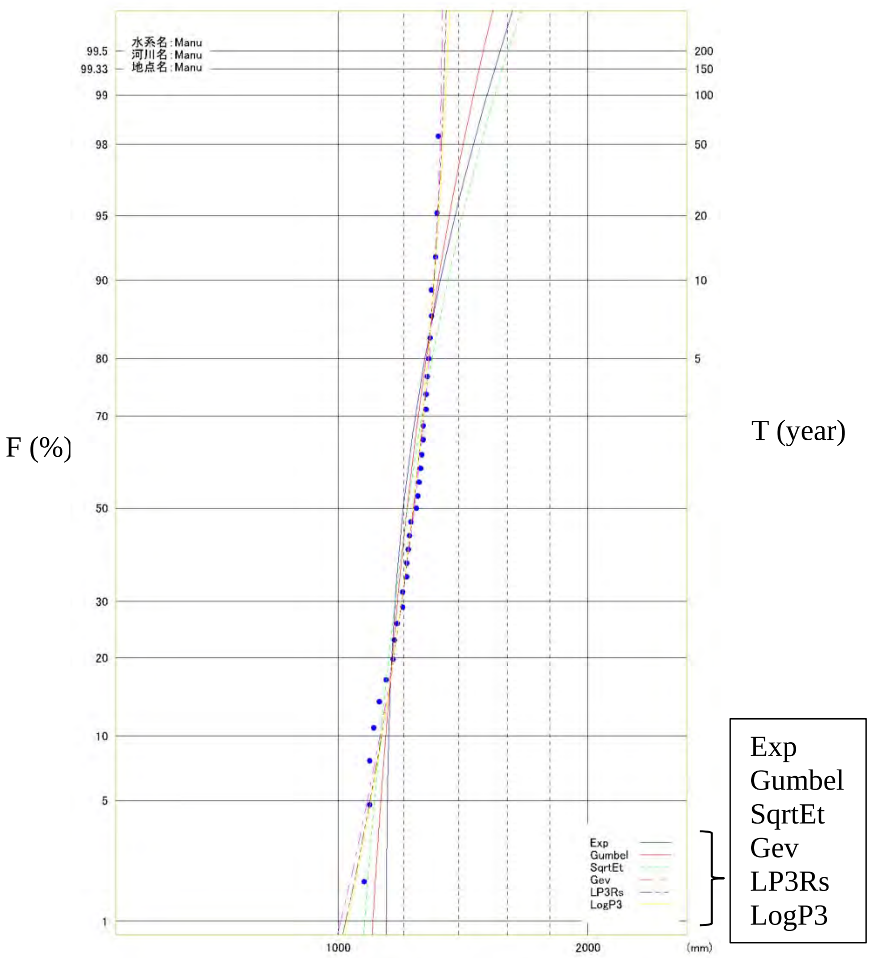
(6) Bias correction and stability evaluation of probable hydrological values

Regarding probability distribution models that satisfies a certain goodness of fit, the jackknife method should be applied in order to correct bias and to evaluate stability of probable hydrological values.

(7) Determination of the probability distribution model

In determining the probability distribution model, among the several models screened by SLSC method, the model of good steadiness judged by the jackknife method should be finally selected as a distribution probability model.

Lognormal probability paper



Annual high water level (mm)

Fig 4 Examples of occurrence probability distribution

2.6.3 Free Board

Embankment free board comprises height of wind set-up (H_w), height of wave run-up (H_z) and safety margin.

In areas where wind set-up and wave run-up do not occur, the free board should be at least 0.9 meters from the view point of safety for inspection at flood time and against attack of driftwoods. However, 1.5m of free board should be provided with embankment of the Jamuna River and Padma River.

In the case of submersible embankments in Haor areas, the nominal free board should be 0.30 meters.

Rises of water level by river bed fluctuation and at bend are involved in Design flood water level.

(Detail explanation)

1. Cyclones and low atmospheric pressures generate wind. High wind brings about rise of water level (wind set-up) in the shallow areas, when it blows toward the land.

   Waves are generated under the condition that winds blow for a long period in the open water area. Significant wave height that represents the height of the waves is estimated from three elements, “Wind direction”, “Effective fetch length” and “Wind speed and Duration”.

   The definitions of those are described in “Guidelines for bank protection; 7.2.4 Waves of JMREMP” in detail. Furthermore, if the directions of the wind and the river flow are the same, shear stress that wind transmits to water surface of river is reduced, because the difference between current speed and wind speed is reduced,

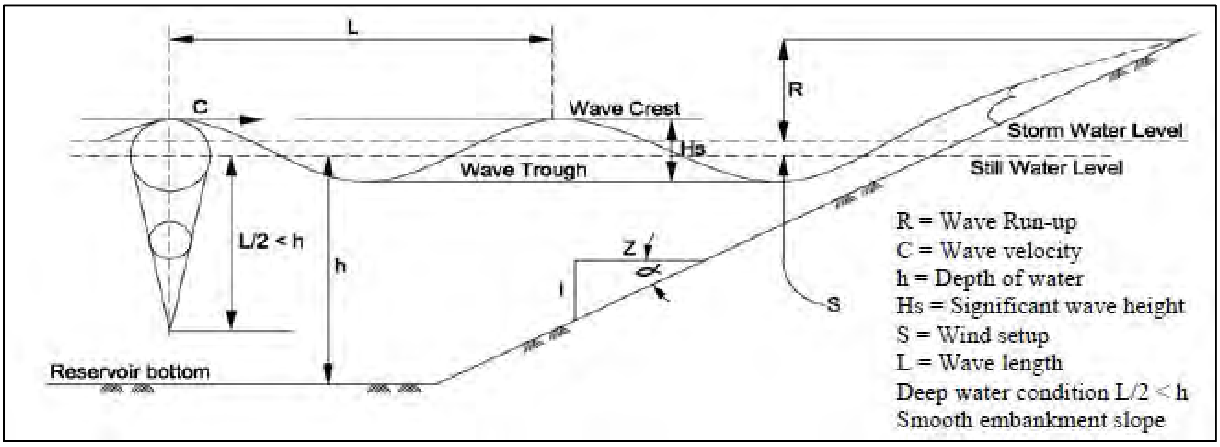


Fig-5 Illustration of wave definitions (Source; Guideline for River Bank Protection)

2. Wind set-up develops with condition that wind blowing lasts for at least 24 hours. A short storm with less duration will hardly generate any significant wind set-up.

   The amount of rise of water level caused by wind set-up is proportional to the square of wind velocity and increases as the water body becomes longer and shallower.

   The amount of rise of water level caused by wind setup is expressed by the following formula:

$$H_w = I_w \times \ell \times \cos \varphi$$

$$I_w = 4 \times 10^{-6} \frac{W^2}{gh}$$ (source; Standard Design Manual, BWDB)

Where, W : Wind velocity (m/s)

g : Gravitational acceleration ($9.81 m/s^2$)

h : Mean water depth (m) along stretch (L)

I_w : Wind setup gradient

ℓ : Length of water area over which the wind is blowing (m)

Φ : Angle at which wind approaches to embankment

3. Waves generated by high winds break when the depth of water is shallow, and water level rises in areas where waves break. When waves break, water level rises on the river side, and this phenomenon is referred to as wave run-up.

The amount of increase in water level resulting from wave run-up is expressed by the following formula:

$Hz = 8\ f\ H \cdot \tan\alpha \cdot \cos\beta\ (1\text{-}B/L)$ (source; Standard Design Manual, BWDB)

* Applicable slope: $1/8 < \tan\alpha < 1/3$

Where, f : Constant

(0.75 [rough rip-rap slope] ～1.25 [smooth asphalt concrete slope])

(1.1 [Vegetated slope])

H : Wave height (m)

α : Angle of embankment side slope to horizontal

β : Angle between direction of incident wave and normal to embankment

B : Berm width of slope on river side (m)

L : Wave length (m)

(1-B/L): This can be omitted if there is no berm

$H = 0.0555(W^2F)^{1/2}$

Where, F : Length of fetch (length of wind blowing region possessing certain wind velocity and orientation; nautical miles)

W : Wind velocity (knots)

$L = T^2 g / 2\pi$

Where, g : Gravitational acceleration ($9.81 m/s^2$)

T : Wave cycle = $(W^2F)^{1/2}$

2.6.4 Extra Banking

Since embankments are often constructed in areas of soft ground and embankments generally comprise earth, subsidence of the embankment due to compression of embankment body and consolidation of foundation ground occurs.

It is necessary to add some extra banking height to crest level of embankment in consideration of the subsidence of the embankment due to the compression of embankment body, consolidation of foundation ground and crest damage caused by weathering and others.

Such extra banking is not included in the design embankment height.

(Detail explanation)

Because embankments are constructed with earth, their construction triggers compression; moreover, when constructed on clayey soil strata, vertical stress is imparted, causing pore water to be discharged from the clay soil strata and consolidation subsidence occurs. Since this causes the embankment to settle and can lead to overflow when flooding occurs, it is necessary to add a corresponding amount to the embankment crest level as extra height (ΔH). Extra Banking does not include compression by compaction.

Normally, 10% of design embankment height (H) is added as an Extra Banking (△H).

Further, subsidence of ground due to groundwater water intake should be taken into consideration at the place where enormous volume of groundwater water intake is made.

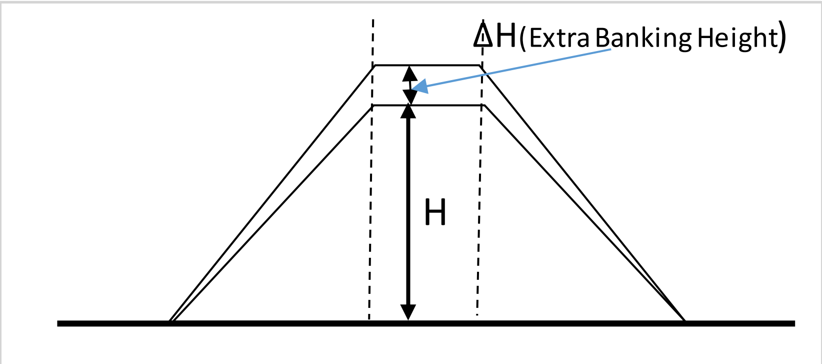


Fig 6 Extra Banking Height

(Reference)

The following extra height is added to the free board in designing embankments in Japan.

Table - 1 Extra Banking Height

Embankment body soil type		Extra Banking Height (cm)			
		Normal soil		Sandy soil	
Ground soil quality		Normal soil	Sandy soil	Normal soil	Sandy soil
Average embankment height	~3m	20	15	15	10
	3~5m	30	25	25	20
	5~7m	40	35	35	30

(Source: Embankment Extra Banking Standard in Japan)

In cases where consolidation subsidence of foundation ground is forecasted in areas of soft ground, subsidence amount should be estimated and the amount should be added to the design crest level. The amount of consolidation subsidence of foundation ground should be estimated through adding up the amount of each layer subsidence as shown below.

$$S = \frac{C_c \cdot H}{1+e_0} log_{10} \frac{P_0+\Delta P}{P_0}$$ (source; Standard Design Manual, BWDB)

S: Amount of subsidence of single layer (m)

Cc: Compression index

H: Consolidation layer thickness (m)

e_0: Initial void ratio

Po: Effective overburden stress before embankment construction (current) (t/m^2)

ΔP :Increased load in each ground layer by embankment (obtained from Fig-7) (t/m^2)

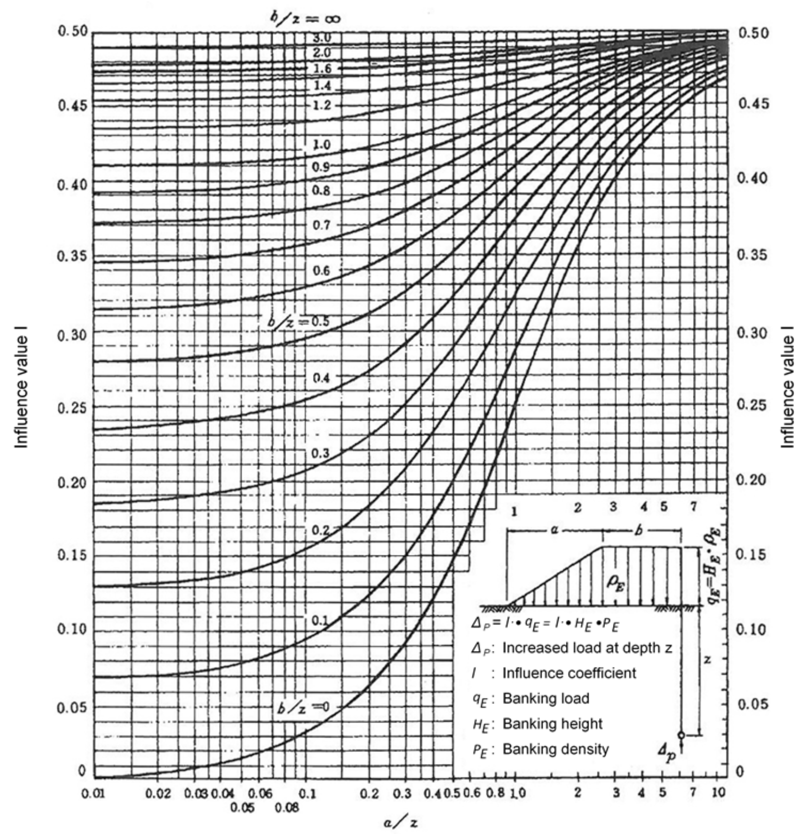


Fig.-7 Vertical Ground Stress Influence Value of Trapezoidal Load (Osterberg Chart)

Further, ΔP also can be obtained by calculation based on the following equation.

$$\Delta P = \frac{q_o}{\pi}\left[\left(\frac{B_1+B_2}{B_2}\right)(\alpha_1+\alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

$$\alpha_1(radians) = tan^{-1}\left(\frac{B_1+B_2}{Z}\right) - tan^{-1}\left(\frac{B_1}{Z}\right)$$

$$\alpha_2(radians) = tan^{-1}\left(\frac{B_1}{Z}\right)$$

*Angles measured in counter-clockwise direction are taken as positive

(Source; Soil Mechanics and Foundations, by Dr.B.C.Punmia)

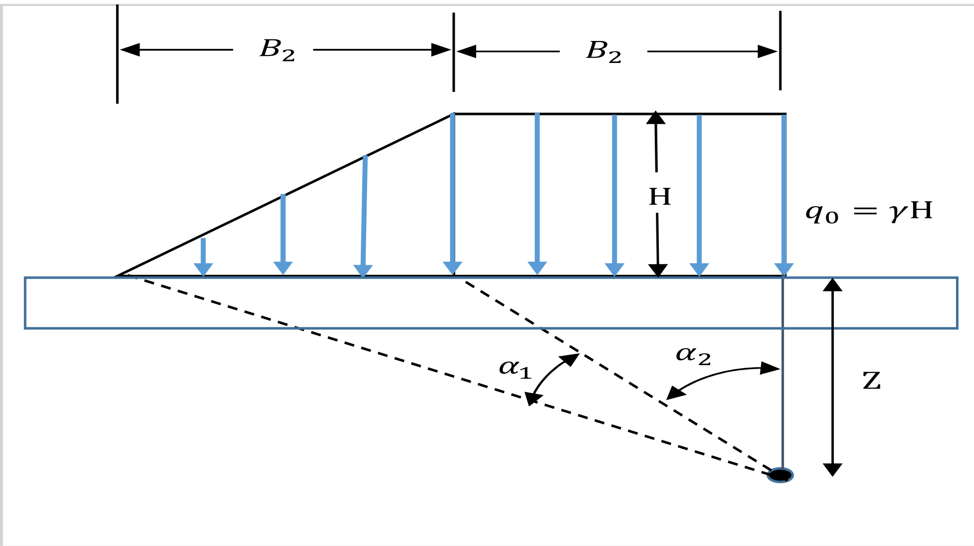


Fig-8 Vertical stress caused by trapezoid of half of embankment

2.7 Crest Width

The embankment crest width, together with the embankment slope gradient, forms the cross sectional shape of the embankment and they are important elements in securing the embankment's safety with respect to seepage.

Meanwhile, since embankment crests are often used as river management roads and public roads, it is also necessary to take such uses into account when deciding the crest width.

(Detail explanation)

From the viewpoint of securing embankment safety with respect to seepage, it is desirable to have a broader crest. On the other hand, if the crest width is made too wide, the embankment will take up more land and reduce precious farmland. Moreover, if an appropriate embankment crest width for road traffic is not adopted, the traffic will cause damage to the crest. It leads to the reduction of road width.

Accordingly, it is necessary to secure the necessary embankment crest width in full consideration of every aspects including traffic utilization.

The embankment crest width should be designed based on the followings;

1) Embankment crest width should be at least 2.50 meters.
2) In cases where the embankment crest is used for management inspection, width of at least 4.30 meters should be secured.
3) In cases where the embankment crest is used as public road, width of 1.0 meter should be added to each shoulder for the use of the public road.

Because embankments in Haor areas are used as community roads during the dry season, it is necessary to secure sufficient crest width on them.

2.8 Side Slope

The criteria for determination of side slope shall be followings; - Embankment slopes should be stable against seepage flow. - Embankment should be stable against shear failure.

(Detail explanation) (Source: Standard Design Manual)

1. Basically side slopes for an embankment (except sea dyke) is decided based on a soil characteristics of embankment body. Generally it should be 1V:2H on the country side and 1V: 2H to 1V: 3H on the river side unless stability considerations requires flatter slopes.

2. Since submersible embankments need to be strong enough to withstand inundation during the flood season and overflow during the period of transition from the pre-monsoon to the flood season, it is desirable to have a slope of 1:3 on both the country side and river side.

2.9 Berm and Borrow Pits

Borrow pits are usually located on the river side from the view point of effective use of valuable land, the haul distance and safety of embankment.

(Detail explanation)

When there is a deep channel close to an embankment on river side in parallel, there is concern that river flow near embankment becomes fast and erodes the embankment slope at times of flooding. Therefore, it is generally preferable to have riverside borrow areas wide and shallow, further in cases of borrow pits are excavated like water channels in the direction of river flow, cross bunds (traverse) should be placed at intervals to ensure that the channel is not too long.

The following regulations should be applied for berms and borrow pits;

(1) The berms i.e. the distance between the river side toe of the embankment and the edge of borrow pit should be between 3.0~10.0 meters, depending on the depth of the borrow pit and height of embankment.

(2) The borrow pit area is preferably wide and shallow, therefore the depth of borrow pits should not exceed 2.0 meters.

(3) The distance between the edge of the borrow pit and the river bank should have a substantial width, be at least 6.0 meters.

(4) In order to prevent increase of flow velocity on the borrow-pit when the river water level is high, cross bunds(traverse) should be provided at every 30 meters of the borrow pit length at right angle to the embankment alignment. The width of cross bund (traverse) should be at least 6.0 meters. Slope of borrow pits should be specified to be gentle to minimize scour from overtopping.

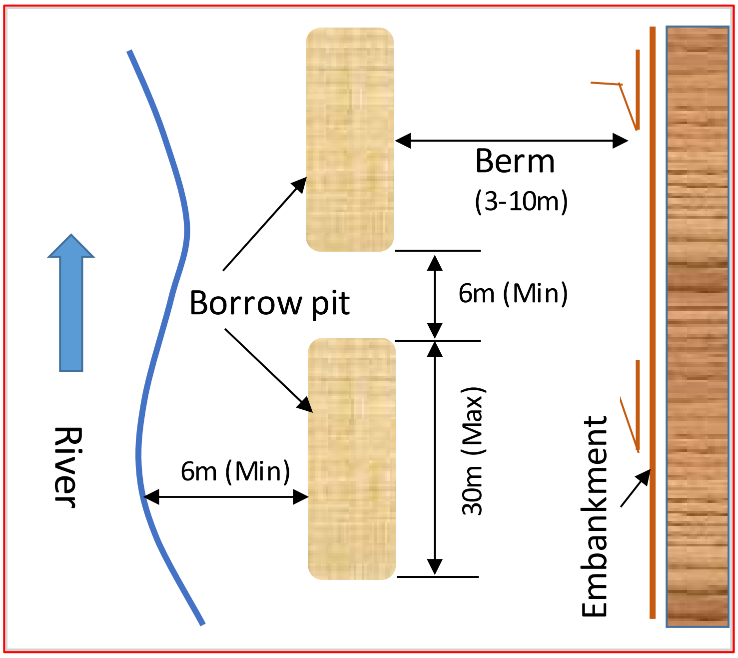


Fig-9 Positional relationship of Berm and Borrow Pits

2.10 Selection of embankment materials

It is desirable for embankment material to satisfy the following conditions.

1) Particle size distribution of material provides high density and large shear strength.
2) Material has high impermeability as much as possible, seepage line is drawn under the toe of embankment.
3) Material has neither characteristics of compressional deformation nor characteristics of swelling, which might have adverse effect on the stability of embankment.
4) Material has high workability and is easy to be compacted.
5) Material is stable against environmental change such as permeation and drying, and accordingly slope sliding and cracks on the surface of embankment is less likely to occur.
6) Material neither contains harmful organic matters nor water-soluble matters.

(Detail explanation)

Desirable soils as embankment material are the followings.

(1) Soil that shows favorable particle size distribution.

It is desirable that soil of various particle sizes are contained in materials of embankment. While coarse materials are effective to strengthen soil, fine materials are also needed to reduce the permeability. The soil contains both of them properly.

(2) Soil that has certain amount of fine materials (defined as soil of less than 0.075mm of particle size) because of ensuring the soil-impermeable.

(3) Soil that does not contain much silt.

Slope sliding is likely to occur in the case that embankment consists of large quantity of silt, because silt has characteristics of reducing shear strength with increase of water content.

(4) Soil that does not contain much amount of fine material.

There is risk of cracks on the surface of embankment in drying when the embankment consists of more than half of fine material.

The followings are measures to improve embankment material in case that soil is not appropriate.

(1) Mixture of other soil

(2) Lowering of water content ratio by drying

(3) Addition of modifiers (lime or cement)

Further, there are some measures other than improvement of embankment material to make up for shortcomings of embankment materials.

(1) Providing layer of impervious materials on the surface of embankment

Embankments along with the Major rivers are likely to be constructed of sand dredged from river bed, because a large amount of embankment materials are needed.

In those cases, surfaces of embankments need to be covered by clay layers with around 0.5m thickness to prevent from erosion by river flow and infiltration of rainfall and river water.

Turf cover or vegetation cover is needed on clay cover layer, because clay is vulnerable to sunshine and rain.

(2) Improvement of cross section (gentle slope of embankment, widening of cross section)

(3) Seepage protection (See 3.2 of this manual)

In Standard Schedule of rates Manual of BWDB, embankment material should consists of more than 30% of clay, 0～40% of silt and 0～30% of sand.

2.11 Compaction

When the river embankment construction, it is desirable to control the compaction degree close to the maximum dry density equivalent to the optimum water content ratio based on laboratory test , so that stability of embankment against infiltration of river water is ensured.

The compaction degree is defined as ratio (%) of dry density of embankment soil at the construction site to maximum dry density by the compaction test at laboratory.

(Detail explanation)

Soil is composed of 3 components, soil particle, water and air which exist in the void. The characteristics of deformation and shear strength are changed by mixing ratio of those 3 components. Stability of soil shall be improved against external forces if soil is compacted in optimum state, because compression efficiency and strength character increase, on the other hand permeability decrease.

The optimum water content ratio is defined as the water content ratio of soil compacted at the state that void of soil became minimum by certain compaction energy. The maximum dry density is that equivalent to the optimum water content ratio. The durability of soil against water infiltration in such state is relatively strong even if condition of compaction changes.

・Water content ratio (w)

$$W = \frac{W_w}{W_s} \times 100 \ \ (\%)$$

・Dry density; $\rho_d \ (g/\mathrm{cm}^3)$

$$\rho_d = \frac{W_s}{V}$$

・Zero Air Void curve

Va =0%,

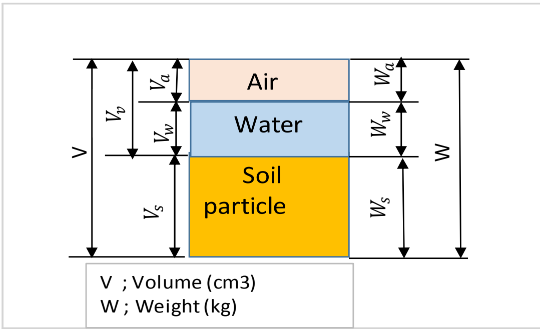


Fig 10 Pattern diagram of soil structure

Sr = Vw/Vv = Ww/Wv (100 %) (Degree of Saturation)

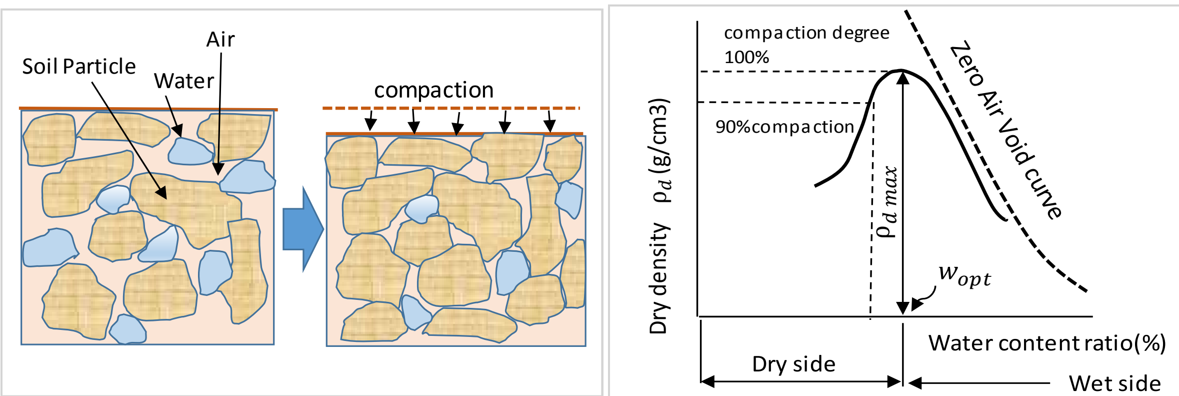


Fig 11 Water content ratio – Dry density curve

River embankment is the structure made of soil to prevent river water from intrusion into land, it is always subject to water infiltration. Therefore, it is desirable that embankment soil is at the state of the maximum dry density and the optimum water content ratio.

The maximum dry density and the optimum water content ratio are examined by compaction test in laboratory at the design stage and the compaction degree is also determined as a control value for embankment stability at the construction stage.

The greater compaction energy becomes, the lower optimum water content ratio and the higher maximum dry density tend to become. The compaction curve in the case of high compaction energy is drawn at high position in Figure-12

Therefore, the compaction test method at laboratory conducted at design stage corresponding to the compaction energy carried out at construction stage should be selected.

Further, Compaction degree is defined as the ration of dry density obtained at site after compaction to maximum dry density obtained in laboratory. In Standard Schedule of rates manual, compaction degree is described to be 85/90%.

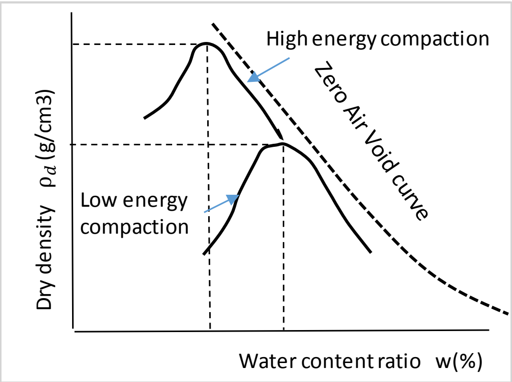


Fig 12 Compaction energy – compaction curve

3. Design specification

3.1 Slope Protection work

3.1.1 General

Embankment slopes and crests must be protected against erosion caused by flowing water, rainfall and waves. Generally the slopes under the design flood level are covered by locally available grass.

However, in places where strong waves and currents are forecasted (at place where necessary set-back cannot be secured or at the outer banks of bends of meandering rivers), revetment works should be implemented in order to protect slopes. Cubic Concrete blocks (C.C.block) with no projection are provided side by side on the slope of embankment for slope protection in Bangladesh.

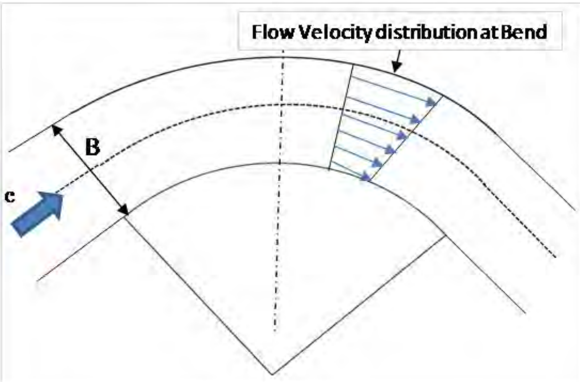
Further, trees should not be planted on berms, crests and slopes, because embankments could collapse when a tree is blown down by the strong wind due to low atmospheric pressure or cyclone.

3.1.2 Projection of block

In the case that blocks with no projections (namely, surface of embankments has no degree of roughness against flow) are provided on the embankment slope, flow velocity of river in the vicinity of the embankments is accelerated. It may lead to failure of embankment on the lower reach where embankments are covered by turf. Therefore, revetment should have certain degree of roughness in general.

Especially, Not only is flow velocity accelerated by centrifugal force at outer bank of bend of meandering rivers, but also secondary flow is generated at the same time. Secondary flow is the flow perpendicular to river flow and it is major cause of local scouring.

Therefore, it is strongly recommended that revetment with degree of roughness should be provided on the surface of embankment slope at outer bank of bend.



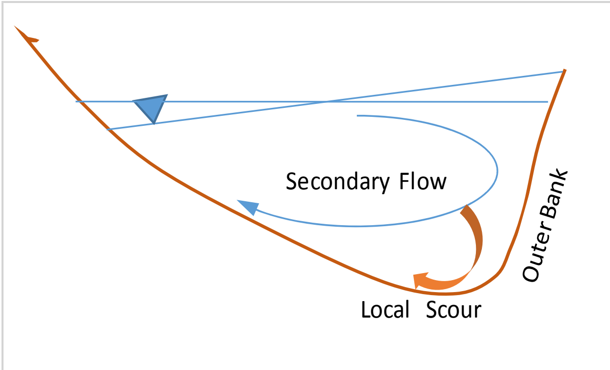


Fig 13 Flow Velocity distribution at Bend

Fig 14 Occurrence of the secondary flow at bend

3.1.3 Design of Revetments for embankment slope

1. Stability of revetment under current attack

Revetment blocks protect embankment slopes from flowing water, while at the same time they are sometimes damaged by sliding and peeling caused by the impact of hydrodynamic forces such as drag and uplift. In cases where the slope gradient is gentler than 1:1.5 and it is not subject to earth pressure and water pressure from behind, evaluation is carried out as described below.

Also, revetment stability with respect to flow is described in the Guideline for River Bank Protection 2010 by BWDB (hereafter referred to as the Guideline) on the assumption that revetment is boulder revetment. Blocks which are generally used for slope protection of embankment is cubic without projection, but it is preferable that cubic block with projection is used at outer bank of bend to prevent secondary flow from erosion of bank. In the case that cubic block with projection is used, stability of block should be examined by referring to the following formula.

《 Evaluation of stability with respect to sliding (in case of dry masonry)》

Stability of block with projection provided on the slope of embankment is assessed by calculation of equilibrium between force of flow and gravity of block acting on block. Fig-15 shows equilibrium of forces.

Stability should be confirmed with the following formula;

$\mu\,(W_w \cdot \cos\theta - L) \geqq ((W_w \cdot \sin\theta)^2 + D^2)^{1/2}$

(Source; Dynamic Design Method on Revetment, by Japan Institute of country-ology and Engineering)

$L= \rho_w/2 \cdot C_L \cdot A_b \cdot V_d^2$ (kgf)

$D= \rho_w /2 \cdot C_D \cdot A_D \cdot V_d^2$ (kgf)

Where; L : Uplift

D: Drag

μ: μ=0.65 (coefficient of friction between soil and suction prevention material)

W_w : weight of blocks in water

θ : Slope gradient

ρ_b : Density of blocks (kgf· s^2/m^4)

ρ_w : Density of water, ρ_w=102 (kgf· s^2/m^4)

g : Acceleration of gravity (9.81m/s^2)

A_b : Projected area of blocks when viewed from above (m^2)

A_D : Projected area of blocks pertaining to drag blocks (m^2)

t_b : Thickness of blocks (thickness not including projection) (m)

C_L : Uplift coefficient

(C.C. blocks with small projection: approximately 0.20)

(C.C. blocks with a little higher projection: approximately 0.15)

C_D : Drag coefficient

(C.C. blocks with small projection: approximately 0.40)

(C.C. blocks with a little higher projection: approximately 0.7)

Note-1; C_L, C_D are determined by experiment results.

Note-2; C_L can be defined to be around 1.0 as a maximum value.

Note-3; C_L, C_D of larger one or Smaller one than original Block don't change if the size of Block is similar to the original one. However, k_S changes in proportion to the thickness of projection height.

ks : Equivalent roughness

(C.C. blocks with small projections: 0.04)

(C.C. blocks with a little higher projection: 0.08)

V_d : Critical neighborhood flow velocity for movement of revetment

$$V_d = \frac{8.5+5.75\log_{10}\left(\frac{t_b}{k_s}\right)+2}{6.0+5.75\log_{10}\left(\frac{H_d}{k_s}\right)} V_0 \ \text{(m/s)}$$

H_d : Design depth of water (when there is a wide set-back, the difference between design high water level and set-back level at the toe of embankment) (m)

H_d is the difference between Design Flood Water level and average river bed level to calculate average flow velocity at the vicinity of embankment.

V_o : Design flow velocity $V_o = \alpha V_m$ (m/s)

V_m : Mean flow velocity (calculated from Manning's formula of average flow velocity)

$V_m = 1/n \cdot H_d^{2/3} \cdot I_e^{1/2}$ (m/s)

n : Manning's coefficient of roughness

α : Correction coefficient (when there is no foot protection of sufficient width on the embankment revetment)

$$\alpha = 1 + \frac{\Delta Z}{2H_d} + \frac{B}{2r}$$

B : Width of river (m)

r : Radius of curvature of bend (m)

ΔZ : Scouring depth (difference between projected deepest riverbed level and average riverbed level) (m)

Estimation measure is described in the Guideline 7.2.5.

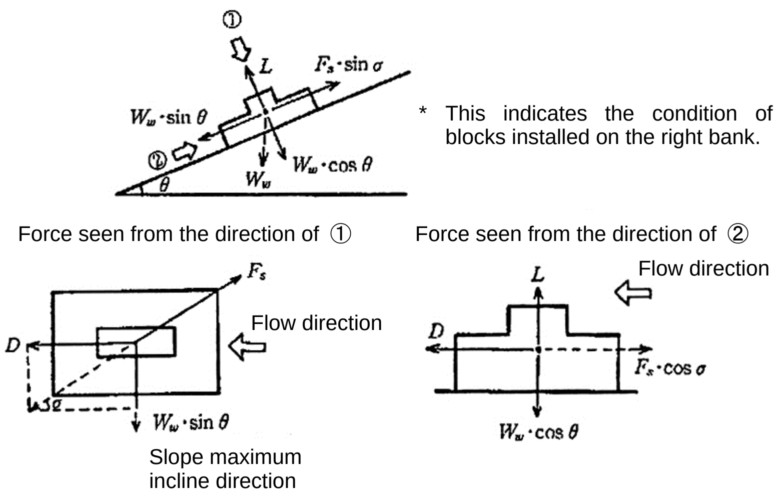


Fig.-15 Model Drawings of Force Balance

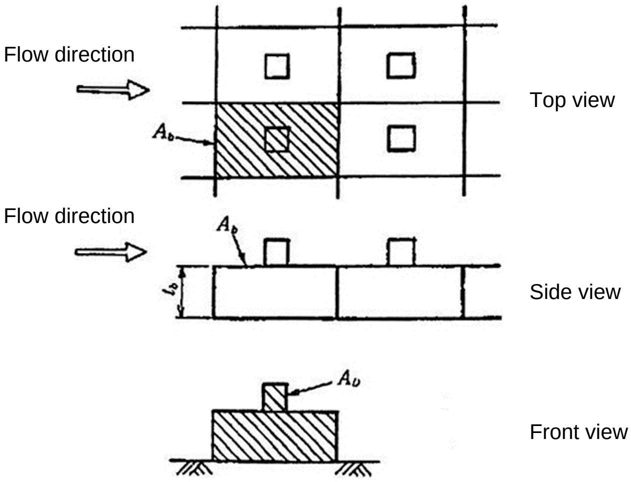


Fig. 16 Explanation of Projected Area

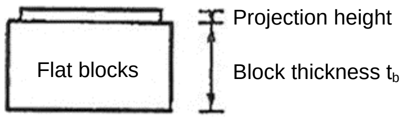


Fig. 17 Definition of Brace Thickness Support t_b

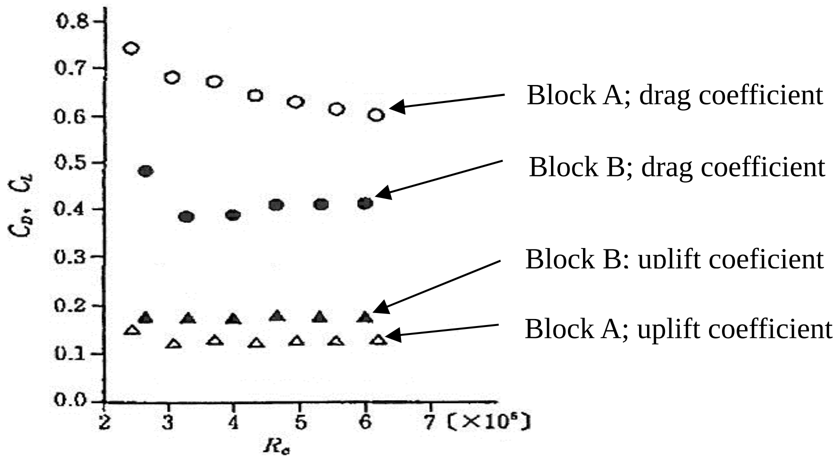
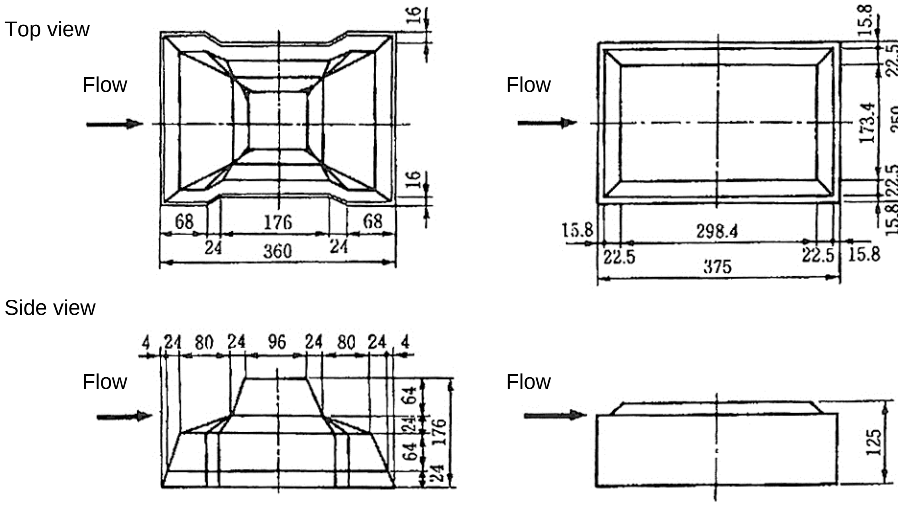


Fig.-18 C_D, C_L of Isolated Blocks based on the experiment

(Here, R_e ; Reinolds Number)



Block A Block B

Fig. 19 Standard Drawing of Block

2. Stability of revetment under wave attack

Embankments in the tidal areas, in Haor areas and along with major rivers, facing large open water, are affected by wave power and wave run-up, depending upon the direction of the wind. Those embankments should be protected by revetments.

In such cases, it is necessary to assess the stability of revetment against wave, necessary weight (size) should be calculated. Many formulas have been proposed until now. Such formulas are described in “Guidelines for river bank protection; 8.4. Stability of Revetment under Wave Attack”.

3.1.4 Block placement with open join

Because the water which permeated into the embankment during the flood is likely to remain in the embankment as a residual water even after the river water level drops sharply at the end stage of flood, it can weaken the embankment and cause the circular sliding of embankment slope on the river side. Therefore, for discharging the residual water, the sediment discharge preventive works such as the geo-textile should be provided underneath the slope protection block, and the slope protection blocks should not be connected by cement each other.

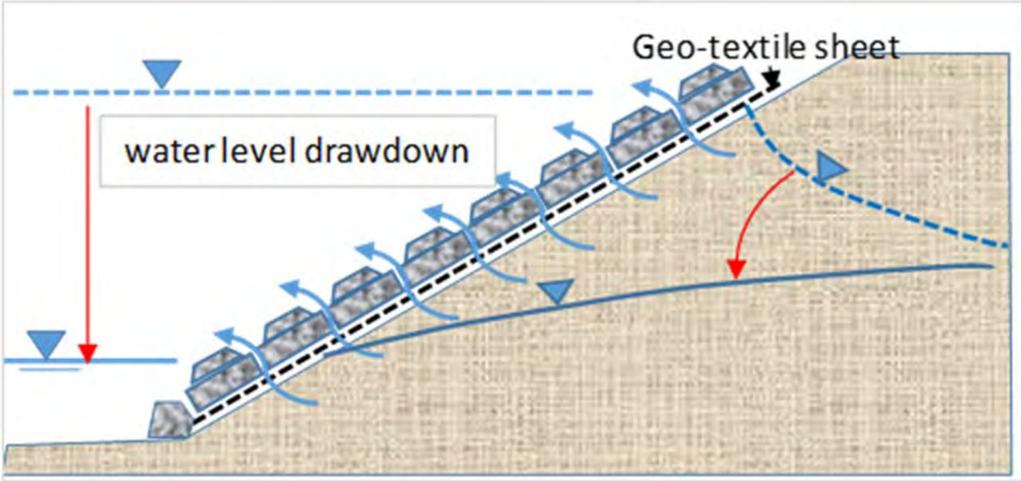


Fig 20.1 Drainage water of residual water from between block with open joint when water level drawdowns

3.1.5 Toe Wall (Foundation work for slope protection blocks)

The slope protection block placed as a countermeasure against the erosion on the slope is designed to stabilize against the flow by the bottom friction between block and the embankment surface. However, it is concern that slope protection blocks could slip down due to the fluctuation of the river flow partially, the toe wall should be provided at the down end of the slope protection blocks not only to support the slope protection blocks and prevent from sliding, but also to protect the foundation from erosion and prevent embankment materials discharge. In the case that there is not sufficient set back distance, the height of the toe wall is decided in consideration with the deepest river bed height of neighboring river bed.

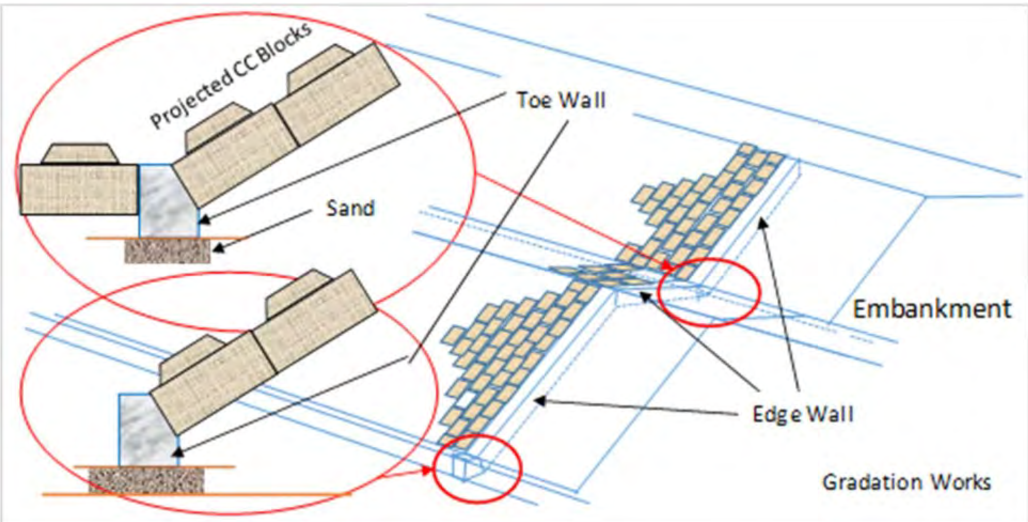


Fig 20.2 Toe Wall

3.2 Seepage Protection Works

3.2.1 General

Seepage protection works should be implemented as needed in order to control seepage and prevent piping phenomena with considering the embankment materials, foundation ground materials, water level and flood continuation time. Approaches for strengthening river embankments with respect to seepage are broadly divided into the following two types, and these are further broken down into various methods.

(1) Methods with respect to the embankment body

1) Cross Sectional Expansion Method: this is intended to reduce the hydraulic gradient inside the embankment body and boost safety with respect to sliding damage.

2) Surface Covering Method: this is intended to prevent and mitigate seepage of rainfall and river water into the embankment body.

3) Drain Method: this is intended to promptly drain rainfall and river water that has infiltrated into the embankment body.

(2) Methods with respect to the foundation ground (intended to mitigate and prevent seepage into the foundation ground)

1) River Side Seepage Control Method: construct sheet piles at toe of embankment on river side in order to prevent seepage from ground.

2) Blanket Method: this is intended to reduce the surface permeability of set-backs.

3.2.2 Methods in Embankment Body for Seepage Protection

(1) Cross Sectional Expansion Method

This is to widen the basic cross section on the river side, thereby extending the seepage length and reducing the average hydraulic gradient, while also improving safety with respect to sliding damage through making the slope gradient gentler than the basic cross section.

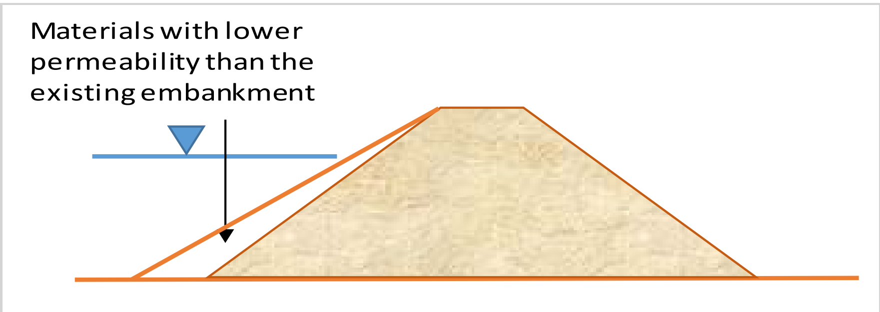


Fig.-21.1 Basic Cross Sectional Shape in the Sectional Expansion Method

(2) Surface Covering Method

This is to cover the slope on the river side with impermeable materials (impervious materials) to control infiltration of river water from the river side slope at flood time. This method is especially effective in cases where the embankment body is composed of sand and impermeable materials are used on the land side. It is also effective for preventing slope on the river side from sliding due to residual water pressure when water level draws down at the end of the flood.

The impervious materials must possess sufficient shear strength after compaction. It is also necessary for the materials to be resistant to deformation and harmful cracking, and to be easy for compaction and to have good workability. Sandy soil containing high fine particle or clayey soil with low water content is suitable for this purpose.

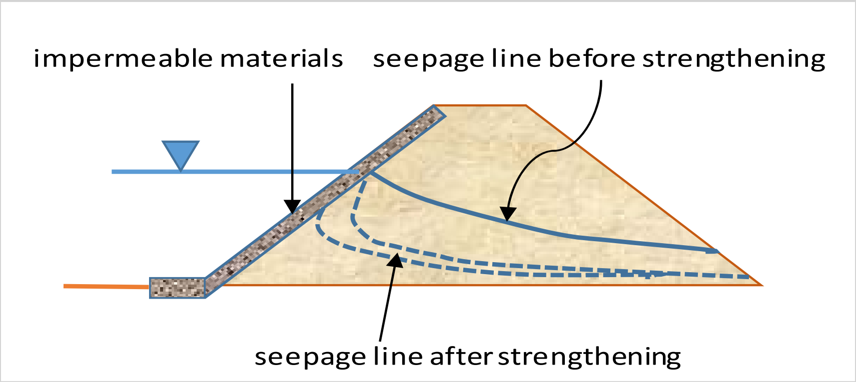


Fig.-21.2 Basic Structure of the Surface Slope Method

(3) Drain Method

This is to collect rain and river water that has infiltrated into the embankment during flooding into a drain at the toe of embankment on the country side and to facilitate prompt natural drainage out of the embankment over the long term. This is mainly intended to lower the seepage line of the embankment body.

The drain work is composed of 1) the drain, to collect the infiltrated water into the embankment foot water channel, 2) the embankment foot water channel, to collect water from the drain to the designated terminal, and 3) the filter, to stop the runoff of soil particles from the embankment body to the drain and to prevent clogging of the drain.

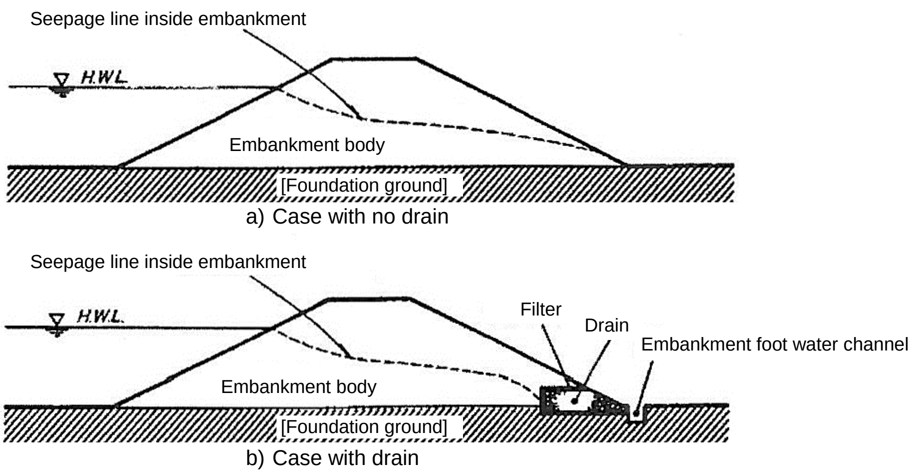


Fig. 22 Structure and Effects of Drain

(Source: Drain Design Manual (2013.6))

1) Width of drain (The length of the direction perpendicular to river flow)

- Width of the drain should be set so that the average hydraulic gradient (H/D) is not steeper than 0.3. According to model experiment by Public Works Research Institute (PWRI) Tsukuba city in Japan, it has been confirmed that there will be hardly any possibility of piping if the average hydraulic gradient is not steeper than 0.3.

2) Drain height (thickness)

- The height of drain is theoretically determined according to the amount of drainage and permeability of the drain. Although it can be fairly thin in general, sufficient cross-section area of flow should be provided and drain height should be at least 0.5 meters in consideration of certainty of execution, and declining drainage function due to subsidence and deformation in the long term.
- The drain bed level is desired to be slightly lower than the ground level to ensure that seepage inside the embankment body is properly drained. However, it is necessary that the drain bed level should not be lower than the bed level of the embankment foot water channel.

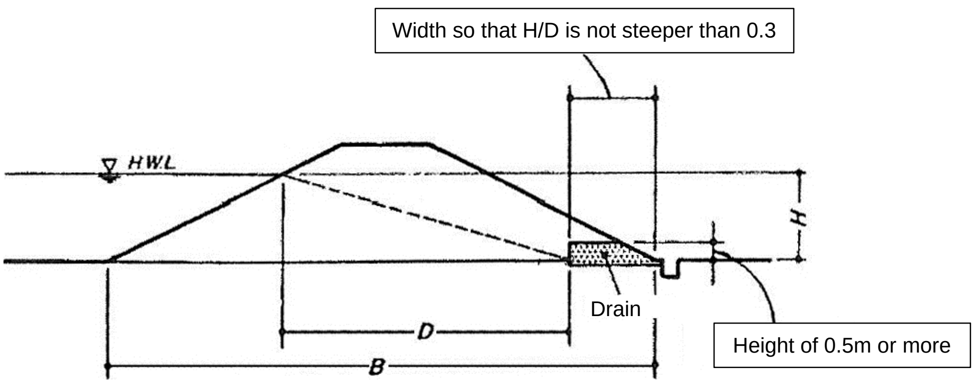


Fig. 23 Drain Cross Section and Mean Hydraulic Gradient

3) Drain materials

- The drain materials must be of such characteristics that seepage water is possibly drained with a small head loss. Accordingly, it is necessary to adopt materials that have a large coefficient of permeability. Materials that have coefficient of permeability around 100 times greater than that of the embankment body are required. Meanwhile, in relation to shear strength pertaining to the stability of embankment, it is necessary to use materials that have an internal friction angle (shearing resistance angle) of not less than 30 degrees as a guide and do not become deteriorated.
- In consideration of the above points, crushed C.C.blocks or bricks are suitable as the drain materials. Moreover, if cage is used for the drain, materials with particle size large enough not to pass through the mesh of the cage must be selected.

4) Filter materials

- The filter, which is used between the embankment body and drain that have very different permeability (particle size composition) with each other, is intended to prevent soil particles of the embankment body from running into the drain with the seepage water, and also to prevent the decline of drain functions due to clogging the drain by the soil particles over the long term.
- Geotextile is frequently used as filter material, and this needs to meet the following conditions;

a. Range of filter opening size : $0.1\text{mm} \leqq O_{95} \leqq D_{85}$

O_{95} : Geotextile 95% opening size (AOS)

(Note) 95% particle diameter of the particle size distribution curve of glass beads that pass through Geotextile

D_{85} : Particle diameter equivalent to 85% of passing weight in the particle size distribution curve

b. It does not become clogged over the long term.

c. Coefficient of permeability: $k \geqq 1 \times 10^{-1}$cm/s ($k \geqq 1 \times 10^{-2}$cm/s at a minimum)

d. Tensile strength (T_P): $T_P \geqq 2.0$ k N/m

e. It is stable with respect to chemical change.

f. It has been undergone hydrophilic treatment.

3.2.3 Method in Foundation Ground for Seepage Protection

(1) Method for shallow permeable foundation

1) River Side Seepage Control Method

The river side seepage control method is to provide an impervious wall comprising clay or sheet piles into the foundation ground around the toe of embankment on the river side, and thereby reducing the quantity and pressure of water permeating the foundation ground from the river and preventing the occurrence of piping. This method is applied in cases where permeable layer is relatively thin.

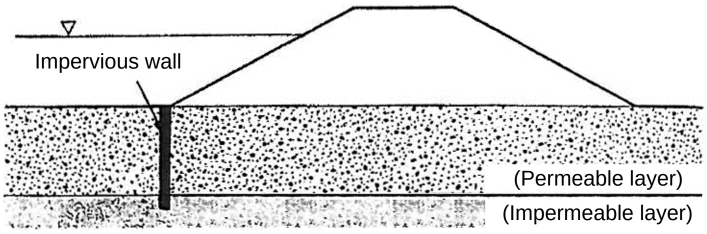


Fig. 24 Basic Structure of River Side Seepage Control Method

2) Center Seepage Control Method

Impermeable soil such as clayey soil is provided at the center of embankment and cutoff trench.

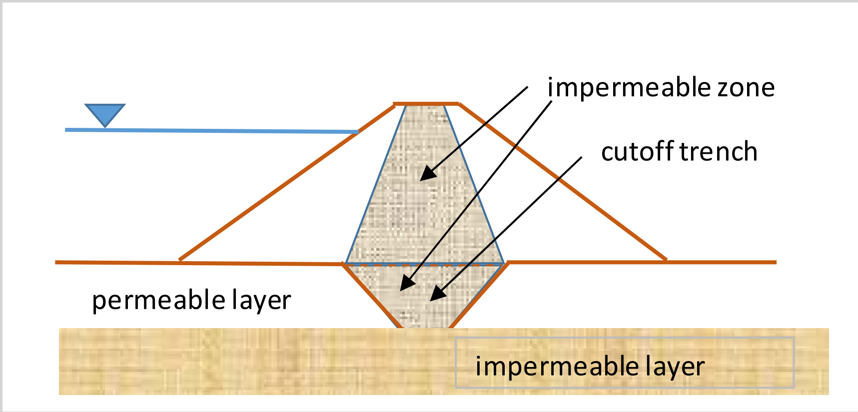


Fig. 25 Center Seepage -cutoff method

(2) Method for intermediate permeable foundation

In the case where permeable layer is relatively thick, steel and bound curtain, slurry trench are provided at the permeable layer.

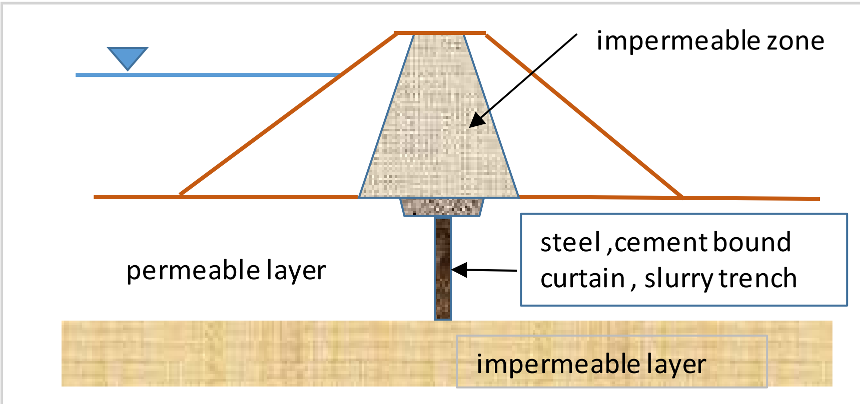


Fig. 26 Steel piling method etc.

(3) Method for deep permeable foundation

In cases where embankment soil is low permeable and material of the set-back is highly permeable, the blanket method is to cover the surface layer above the set-back on the river side with impervious materials, extending the seepage length to reduce seepage pressure in the foundation ground, and thereby improve stability of the country side slope toe with respect to seepage.

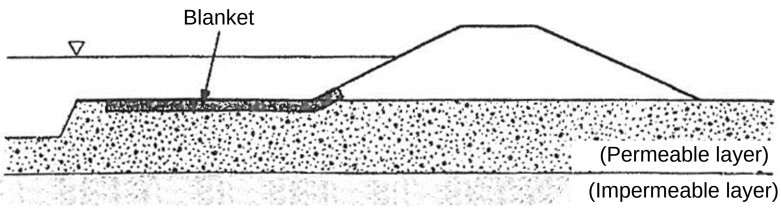


Fig. 27 Basic Structure of the Blanket Method

The length of blanket should be decided by conducting seepage flow calculation and sliding stability calculation and reviewing safety with respect to sliding and piping. The saturated coefficient of permeability of impervious materials needs to be not higher than 1 x 10^{-5}cm/s.

3.3 Toe Protection Works and Foot Protection Works

In cases where hardly any set-back can be secured and the embankment is almost adjacent to the river bank, toe protection works should be provided in order to protect the toe of embankment from erosion, and foot protection should be provided in order to prevent the riverbed scouring at the toe of embankment.

Toe protection works are to drop C.C. blocks (cubic concrete blocks) or geo-backs from the water surface onto the embankment toe slope in order to form a protective layer over the slope under water, while foot protection works are to prevent the riverbed scouring by placing C.C.blocks, geo-bags, riprap, gabion or mattress on the river-bed. Work types should be selected in consideration of water level during construction (dry season).

C.C.blocks or Geo-bags are usually provided for Foot protection work. Those are also placed on the place in front of toe of embankment where erosion or scouring is forecasted. Those will be the Toe protection of embankment, named Falling Apron, so that they form a protective layer on the toe slope when it becomes eroded and scoured in the future. Further information on these methods, refer to “the Guideline for River Bank Protection; 8.7 Design of Toe Protection”.

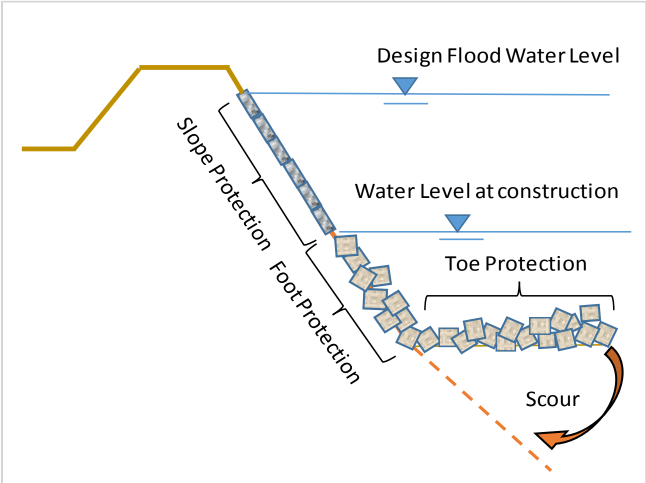


Fig-28.1 Toe protection and Foot protection

3.4 Transition zone works and Edge Wall

In case that new embankment and adjacent existing embankment differ in sectional shape, sectional profile in the transition zone between the new and existing embankments should be changed smoothly in order to avoid occurrence of eddies due to sudden change of profile. Moreover, roughness in the transition zone should be also changed smoothly. The slope protection works in the transition zone should be with flexibility and appropriate roughness, such as C.C.blocks with projection, etc.

In addition, the slope protection work should be provided with the edge walls at the both ends to secure the stability of slope protection blocks against erosion.

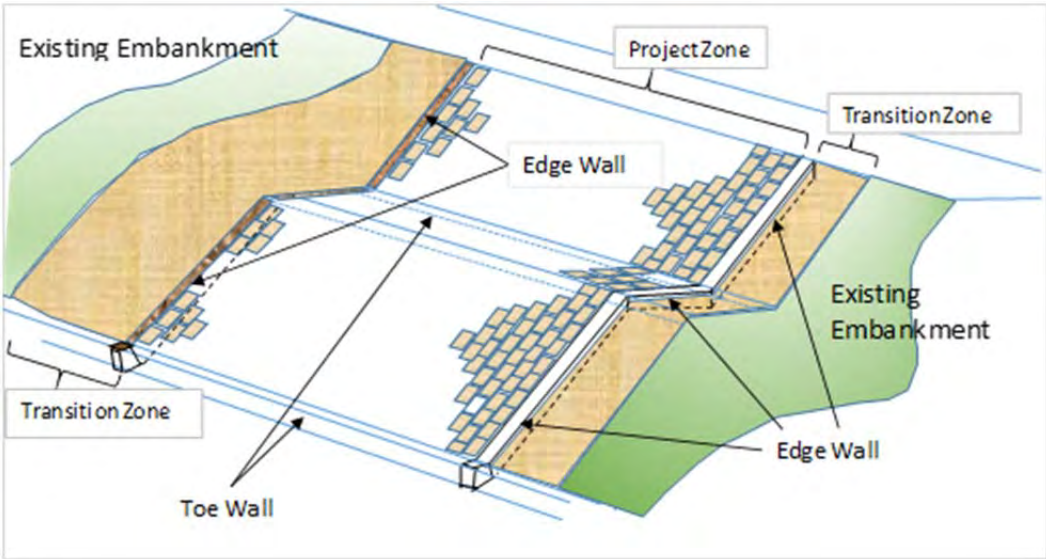


Fig. 28.2 Transition zone works and Edge Wall

3.5 Design of Crossing Structure of Embankment

Sluice gate and Water gate are concrete structure with rigid foundation, support pile foundation is used to prevent from the subsidence of the structures. On the other hand, Embankment is a soil structure constructed on the ground directly.

For this reason, it is difficult to adhere the joint portion of the embankment compactly with the structure for a long period of time. There are likely to be seepage paths at the joint portion and cracks at crest of embankment. Those will be a weakness against flooding.

Particularly in areas where the foundation ground is soft ground, cracks and seepage paths are likely to occur in the embankments around the structure due to uneven settlement of structures and embankments. Those are leading to failures of embankment during flooding.

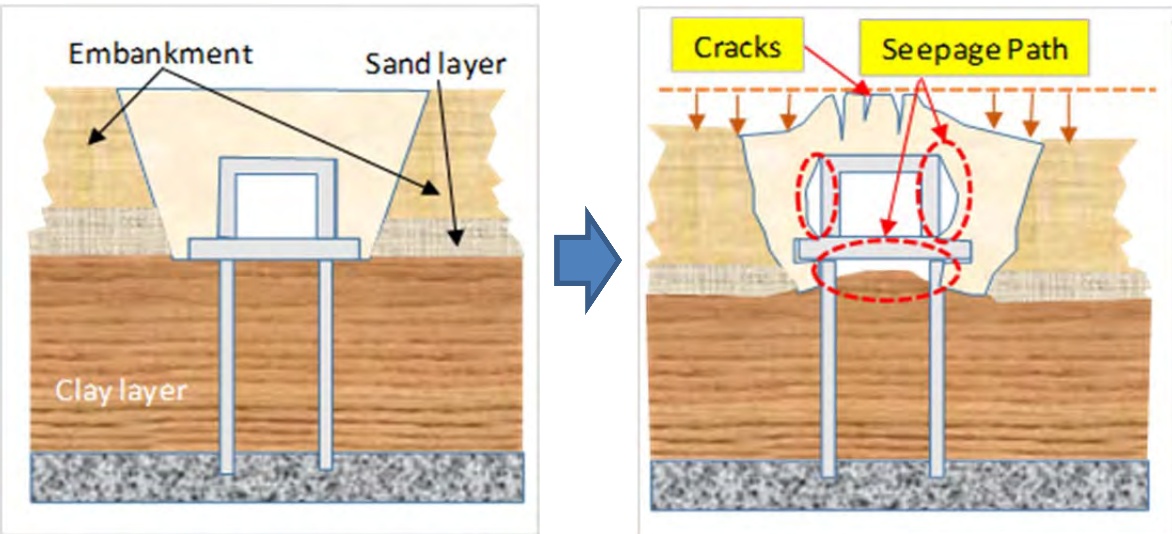


Fig. 28.3 Cracks and Seepage Path around Sluice gate

In the junction between embankment and structure, cracks and seepage paths along the structure are likely to be developed due to difference in consolidation between the surrounding embankment and the foundation ground of structure.

If the seepage flow increases due to development of cracks and seepage paths, the Soil material of embankment and foundation ground will be sucked out, which may lead to collapse of the embankment.

In particular, cracks is easily developed right under the bottom plate of the structure, cracks and loose compaction area inside the backfill soil, and uneven and cracks at the embankment crest.

In order to prevent above "piping", the crossing structure shall be constructed with "cut-off" wall having an appropriate length of 1 m or more in width. The cut-off wall prevents piping phenomena by prolonging the creep length and lowering the seepage pressure of the structure. In case of wider embankment and longer crossing structure, it is necessary to provide two or more cut-off walls.

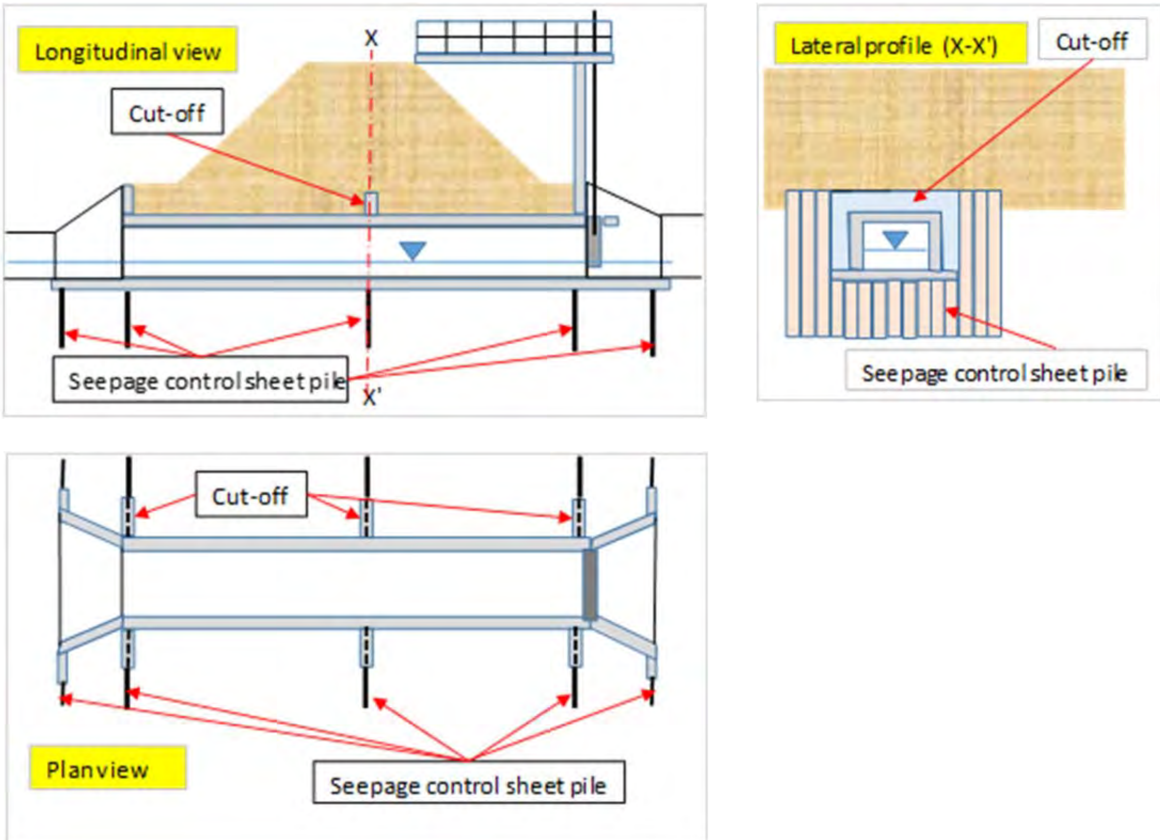


Fig. 28.4 Cut-offs and Seepage control sheet piles of Sluice gate

4. Verification of Embankment Safety

4.1 General

The embankment cross section is usually determined based on the design standards described in section 2 above, however, because embankments are basically made of earth, it is sometimes not possible to maintain the necessary crest level, crest width, design slope etc. due to erosion, seepage, earthquake, and shortage of bearing capacity of foundation ground.

Accordingly, when constructed in places where such risks are expected, embankment safety should be verified by examining the soil characteristics of embankment and foundation ground and others.

4.2 Verification of Embankment Safety with respect to Seepage

4.2.1 Embankment failure caused by Seepage

Embankment failure caused by seepage is broadly divided into sliding damage arising from seepage to the embankment body, and piping damage in foundation ground and embankment body.

In the flood season, the initial rains cause saturation in the embankment body, and the rising river water level causes the seepage line to rise inside the embankment. As a result, pore water pressure inside the embankment body increases, the shear strength of soil decreases, and finally sliding occurs on the country side slope of the embankment. Moreover, when the water level falls towards the end of the flood, seepage water stays inside the embankment body, greatly destabilizing the slope on the river side. Accordingly, when constructing embankments, it is important to carry out sufficient compaction to increase the shear strength of soil in the embankment body.

Moreover, in the case that river water level is high and lasts long, river water continuously permeates the embankment body or foundation ground, and when the hydraulic gradient of seepage water becomes steep enough, seepage force causes soil particles to move and there is a possibility that piping will occur around the toe of embankment on the country side.

In consideration of the above points, when deciding the shape of embankment, the followings should be confirmed by means of phreatic line (seepage line) drew by graphic solution and circular sliding analysis. If the prescribed cross section of embankment is verified not to be stable, it is necessary to change the design elements; e.g. to make the gradient of embankment slopes gentler.

1) Embankment slope on the country side must be stable against sliding and piping when the river water level stays at the design flood level for a prolonged period.

   If it is not stable against sliding, embankment slope should be gentler, otherwise widening of crest or filling on slope of on the country side should be conducted. ($F_s \geqq 1.5$)

   If it is not stable against piping, embankment slope should be gentler or drain filter should be provided at toe of embankment on the country side.

2) Embankment slope on the river side must be stable against sliding when the river water level rapidly drops towards the end of the flood ($F_s \geqq 1.5$)

3) It is preferable that the seepage line is within the embankment body so that no piping from the slope takes place. Even if seepage line is above toe of embankment on the country side, limiting Value of Q (amount of runoff) should not exceed 1.0m^3/day/m. Filter (drain) should be provided in the case that Q exceeds 1.0m3/day/m.

In addition to that, piping is unlikely to occur in the case that embankment soil is consist of clay.

4.2.2 Seepage Line and Piping

The seepage line (or saturation line) is defined as the boundary where positive hydrostatic pressure occurs in the embankment cross section, and hydrostatic pressure at the seepage line is zero, equivalent to the atmospheric pressure. The seepage line denotes the boundary between dry soil and wet soil, and in calculation of soil shear strength, soil above the seepage line is regarded as dry while the soil below is regarded as wet.

In order to conduct stability analysis of embankments by means of the graphic solution, the most important thing is to determine the seepage line. In deciding the seepage line by the Casagrande method, the seepage line is drawn as follows by treating it as something close to the base parabola.

In this figure (Figure 29), corrected length (Δ a) and seepage length (a) of the seepage are given by the following formula;

(Source; Irrigation Engineering and Hydraulic Structures, by Santosh Kumar Garg)

Base parabola is defined as following formula.

$$X = \frac{y^2 - S^2}{2S}$$

Where; $S = \sqrt{H^2 + b^2} - b$

$$\Delta a = (a + \Delta a)\,\frac{180^\circ - \alpha}{400^\circ}$$

α: Angle formed by the horizontal surface and seepage discharge face.

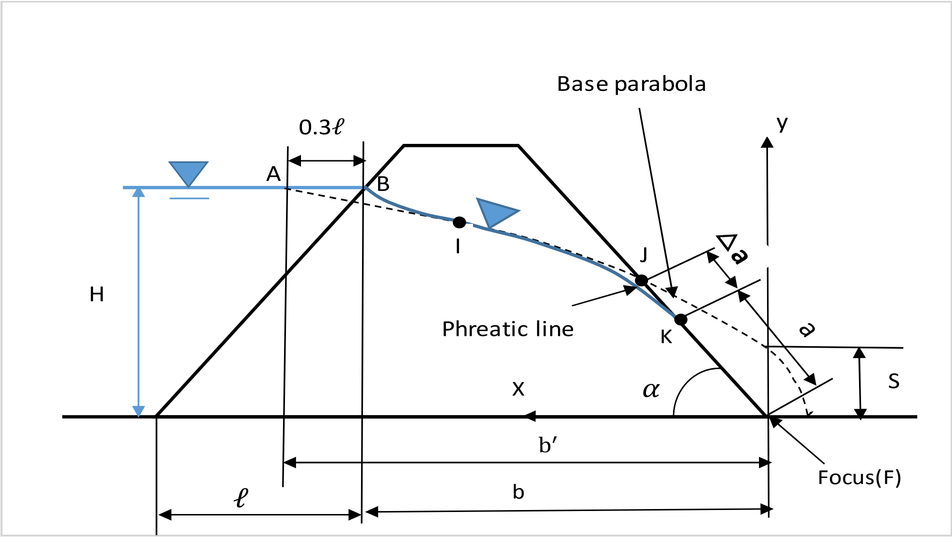


Fig. 29 Seepage Line and Seepage Length

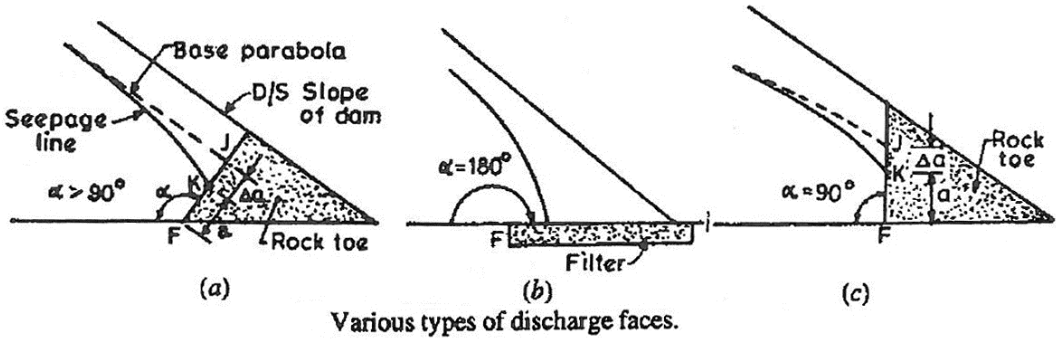


Fig. 30 Cases of Seepage Water Discharge Faces when Drain is provided

Here, the amount of runoff Q will be as follows:

Q = k ・S (k : Coefficient of permeability)

S = $\sqrt{b^2 + H^2} - b$

Here, the runoff length (a) in the case of no drain is given by the following formula.

① When α is no higher than 30° (when there is no seepage prevention drain)

$$a = \frac{b'}{\cos\alpha} - \sqrt{\left(\frac{b'}{\cos\alpha}\right)^2 - \left(\frac{H}{\sin\alpha}\right)^2}$$

② When α is 30°~ 60°

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2 \cot^2\alpha}$$

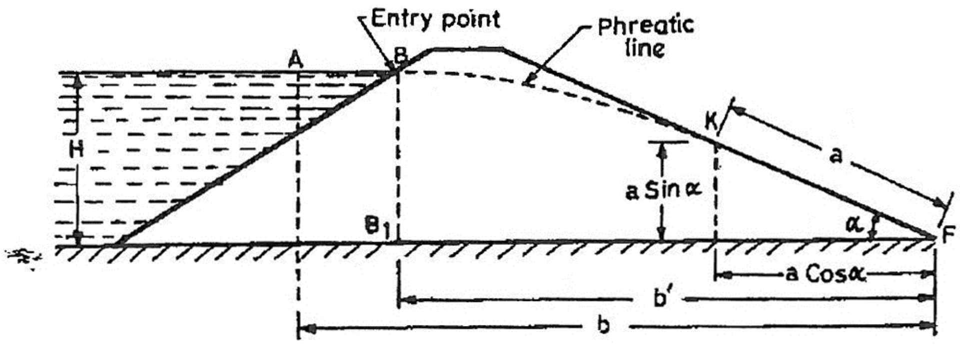


Fig. 31 Seepage Length when there is no drain

4.2.3 Circular Slip (Sliding)

Generally, when slopes collapse, a large lump of earth often slips along a circular slip face.

The embankment slope stability is determined by the Swedish circle method (Fellenius's graphical analysis) or the Bishop method derived from the rotation moment equilibrium formula of “force that causes sliding” and “force that resists sliding” in the slip face.

This method involves dividing earth lumps in a assumed slip circle into ‘n’ slices in the vertical direction and, focusing on the moment equilibrium of force in each slice, determining stability of the slip face according to the ratio of total force causing the slices to slip and total force preventing slip (F_s: safety factor).

Description in detail is as follows. Upon setting the slip face after assuming the center and diameter of the slip circle, earth lumps in the assumed slip circle are divided into ‘n’ slices in the vertical direction and the ratio of total force causing each slice to slip and total force preventing slip (F_s: safety factor) is sought. Safety factors should be calculated for multiple assumed circle centers and it should be confirmed whether or not the minimum safety factor exceeds the criteria (F_s ≧1.5). These calculations should really be implemented by computer.

When considering the seepage line, the safety factor should be calculated by the following formula. Here, this formula ignores forces acting between slices.

$$F_s = \frac{\Sigma\{cl + (W - u \cdot b) \cdot cos\alpha \cdot tan\varphi\}}{\Sigma W \cdot sin\alpha}$$

(Source; Guidance structure study of the river embankment, by Dynamic Design Method, by Japan Institute of country-ology and Engineering)

Fs : Safety factor
u : Pore pressure of the slip face (kPa)
c : Adhesive force of soil along the slip face (kPa)
l : Length of slice circle (m)
φ : Internal angle of friction of soil along the slip face (°)
b : Width of slice (m)
W : Weight of slice (kN/m)
ρ_w : Unit volumetric weight of water (kN/m^3)
α : Angle of inclination of the slice base (°)

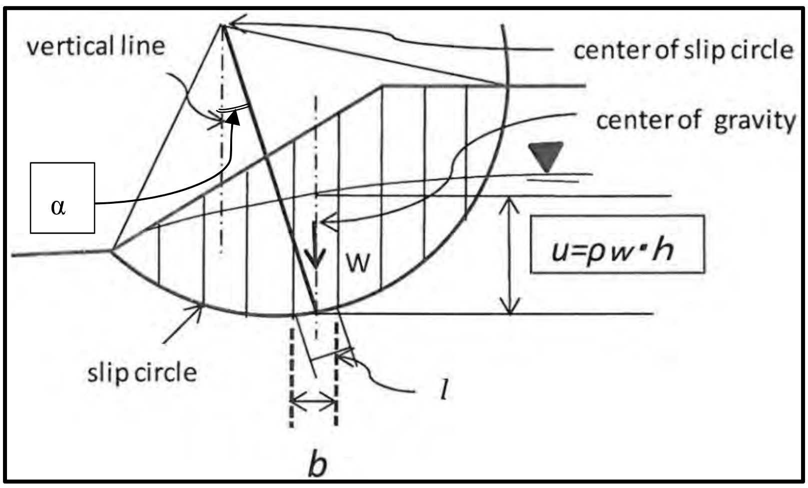


Fig. 32 Slip Circular Calculation based on the Swedish circle method

(Note) Pore water pressure working on the slip face, caused by seepage and drainage, is considered. Change in pore water pressure caused by volumetric expansion of soil in line with shearing is not considered because this is reflected in values of C and ϕ based on soil testing .

Table-3 Strength parameters to the soil quality.

Slice base conditions	Strength parameter
Clayey soil with low permeability [Undrained condition: CU test]	Cohesion: C_u angle of internal friction $\phi \fallingdotseq 0$
Sandy soil with high permeability [Drained condition: CD test]	Cohesion: C_d , $C_d \fallingdotseq C'$ angle of internal friction $\phi_d \fallingdotseq \phi'$

(Note-1) C',∅': If consolidated drained triaxial compression test (CD test) cannot be implemented due to huge time required for the test, values C', ∅' obtained by subtracting pore water pressure from the result of consolidated untrained triaxial compression test (CU test) may be used instead of Cd and $\emptyset_d$.

(Note-2) In the case Clayey Soil, $q_u/2$ (unconfined compression strength from unconfined compression test) may be used substituting the value Cu obtained from CU test.

(Note-3) In the case Sandy Soil, Cd,$\emptyset_d$ (shear strength from direct shear test) may be used instead of the values from CD tests or CU tests

Further, there are some cases that it is impossible to secure sufficient set back distance at outer bank of bend. In such case, it is necessary to verify the safety of embankment sliding at not only the highest flood water level, but also the end of flooding, because it is assumed embankment slope could slides on the river side due to the remaining of water that infiltrated during flood time in such case that there is no sufficient set back.

In case of circular sliding calculation at the end of flood to verify safety of embankment, seepage line in the embankment body at highest flood water level can be adopted for the seepage line at the end of flood from the practical point of view, however, as for the river water level, that at the end of flood should be used. Because only seepage line at steady state is obtained by graphic solution.

4.3 Verification of Embankment Safety with respect to Erosion

It is specified that river embankments should be covered with vegetation. The erosion resistance of vegetated slope against flowing water exists in the roots (piliferous layer) in the case of vegetation like turf-grass species with low plant length. It is thought that the piliferous layer mitigates tractive force through slowing flow velocity near the ground surface.

Concerning the erosion resistance of lawn grass, when the permissible depth of erosion is assumed to be 2.0 centimeters, the relationship between friction velocity (U_*), quantity of root hair (σ_o) and duration of water level higher than set-back height (t), which expresses the shearing force of slope, is shown in the following figure. The figure is used under following conditions

1) Plant communities in which grasses (root diameter is roughly less than 1 mm) are the predominant species, and there is hardly bare ground.
2) Slope materials are composed of silt.
3) The average quantity of root hair is in the range of 0.02~0.12gf/cm$^{3.}$

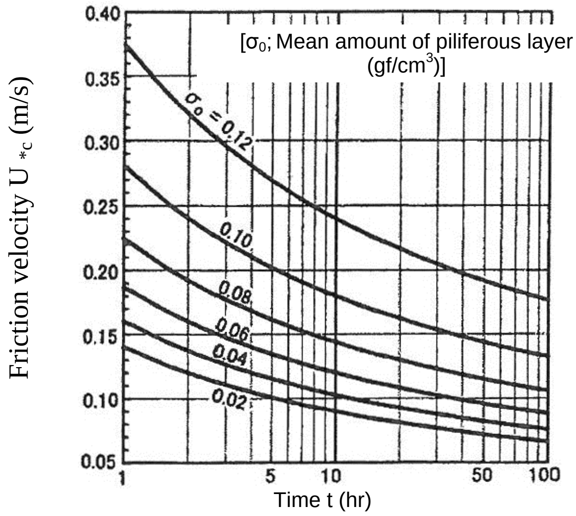


Fig. 33 Surface Erosion Resistance of Vegetation
(relationship of root hair length and friction velocity)

(Note) Friction velocity: U * =$\sqrt{g * H_d * I_e}$

g : Gravitational acceleration (9.8m/s^2)

H_d : Design water depth (in the case of a wide set-back, the difference between the design water level and set-back level) (m)

I_e : Energy gradient ≒ Riverbed gradient

Roughly speaking, when the representative flow velocity is less than 2m/s, erosion -resistance on the embankment river side slope can be deemed to be secured.

In the place that the velocity in the vicinity of embankment exceeds allowable flow velocity mentioned above, embankment slope on the river side should be protected by revetment, especially embankment slope at outer bank side of bend should be protected by revetment, and further foot protection works also should be provided because local scouring takes place due to accelerated flow and secondary flow.

4.4 Verification of Embankment Safety with respect to Earthquake

4.4.1 General

Safety examination of embankment with respect to earthquake has not been conducted in Bangladesh, and there are no applicable standards. Moreover, there have been no reports on embankments sliding due to a result of inertial force or foundation ground subsidence caused by liquefaction during earthquakes.

Even if sliding may have occurred as a result of inertial force during earthquakes in the past, it did not occur on a large scale and didn’t require much cost or manpower to repair. Also, because embankments in Bangladesh generally have low heights (only 4~5 meters even on major rivers), even if liquefaction did occur in the past, the resulting subsidence was not large enough to cause major damage to surrounding areas.

In this manual, basic policy and outline of verification and relevant countermeasures with respect to earthquakes based on Japanese experience is introduced as follows in consideration of the possibility of urbanization around embankments and necessity of safety examination in future.

4.4.2 Basic policy

Subsidence of embankment arising from liquefaction is the most predominant damage in Japan. According to the past experience in Japan, there have been no cases where embankment was completely lost, and a certain portion of embankment height (around 25% at least) always remained.

In areas where the level of land behind the embankment is rather low than the river water level when earthquake occurs, it is possible that the river water will overflow the damaged embankment and cause secondary disaster such as massive flooding of the hinterland.

In Bangladesh, it is likely that river water level is kept to be considerably higher in flood, it is necessary to examine the safety of hinterland against flooding in case of earthquake, and the resistance of embankment against earthquake should be verified.

Resistance of embankment against earthquake, rather than the embankment is not broken, is that significant subsidence of embankment which causes flooding in hinterland does not occur even if earthquake happens. In other words, it is whether height of embankment to prevent from flooding at the time of earthquake is maintained, although subsidence of embankment occur.

4.4.3 Determination of potential for liquefaction

Evaluation on liquefaction potential is conducted for the soil layer existing in 20 m or less below the ground surface based on the following 2 criterions as a preliminary screening. When all of these 2 criterions are satisfied, the ground is evaluated as having the potential of suffering liquefaction.

1) The ground water level is 10 m or less below the ground surface.
2) The soil layer has FC (fine particle content) of not higher than 35%, or even if it is more than 35%, the plasticity index is 15 or less.

4.4.4 Liquefaction judgment and embankment stability calculation

(Source; Guidance structure study of the river embankment, by Dynamic Design Method, by Japan Institute of country-ology and Engineering)

In cases there is a possibility of liquefaction in (2) above, the following "Resistivity to liquefaction (F_L)" should be calculated layer by layer of soil, and if this is 1.0 or less, liquefaction could occur.

$F_L = R / L$

Where F_L; Resistivity against liquefaction
R; Dynamic shear strength ratio
L; Seismic shear stress ratio

Here, R, L are calculated from the soil "N value", fine particle content (passing mass percentage (%) of soil particles with diameter size of 75µm or less).

○ $R = C_w \cdot R_L$

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} & (N_a \geqq 14) \end{cases}$$

(In case of Sandy Soil)

$N_a = C_1 \cdot N_1 + C_2$

$N_1 = 1.7 \cdot N/((\sigma_v'/98.0) + 0.7)$

$$C_1 = \begin{cases} 1 & (0\% \leqq FC < 10\%) \\ (FC+40)/50 & (10\% \leqq FC < 60\%) \\ FC/20-1 & (60\% \leqq FC) \end{cases}$$

$$C_2 = \begin{cases} 0 & (0\% \leqq FC < 10\% \\ (FC-10)/18 & (10\% \leqq FC) \end{cases}$$

Where C_w; the correction factor due to seismic motion characteristics ($C_w = 1.0$)

R_L; Cyclic tri-axial shear strength ratio

N_1; N value equivalent to Effective load pressure 98.0kN/m^2

N; N value

N_a; Correction factor that takes into account the effect of particle size

σ_v; Total stress

σ_v'; Effective stress (=Total stress - buoyancy force)

C_1, C_2 ; Correction factor of N values by the fine fraction content

FC; Fine fraction content (%)

(Passing mass percentage of particle below 75μm)

○ $L = r_d \cdot K_s \cdot \sigma_v/\sigma_v'$

$r_d = 1.0 - 0.015$ x

$\sigma_v = \rho_{t1} \cdot h_\omega + \rho_{t2}(x - h_\omega)$

$\sigma_v' = \rho_{t1} \cdot h_\omega + (\rho_{t2}' - \gamma_\omega)\ (x - h_\omega)$

Where r_d; Reduction factor in the direction of the depth of the seismic shear stress ratio along with the depth

K_s; Design seismic intensity for liquefaction

(Ks in Bangladesh are shown in Fig 29)

x; Depth from the ground surface (m)

ρ_{t1}; Density of soil in shallow than water table position (kN/m^3)

ρ_{t2}; Density of soil in deeper than the water table position (kN/m^3)

γ_ω; Density of water (kN/m^3)

h_ω; Depth from the ground surface (m) to the ground water level

4.4.5 Calculation of subsidence amount

There is no calculation method of subsidence amount of embankment after an earthquake. The subsidence amount is actually estimated from the safety factors for stability of embankment calculated based on the circular slip method. The relationship between the safety factors for stability of embankment and the subsidence amount is derived from previous statistical data, and shown in Table 4 .

There are two methods to calculate the safety factor for stability of embankment.

1) The circular slip method that takes into account only the excess pore water pressure (Δu method)

2) The circular slip method that takes into account only the inertia force (K_h method)

a. $\Delta \underline{u}$ Method

○ $F_{sd}(\Delta u) = \dfrac{\Sigma[c \cdot l + (W - u \cdot b - \Delta u \cdot b) \cdot \cos\alpha \cdot \tan\emptyset]}{\Sigma W \cdot \sin\alpha}$

○ $\Delta u = \begin{cases} \sigma'_V & \text{(in case of } F_L \leqq 1.0\text{)} \\ \sigma'_v \cdot F_L^{-7} & \text{(in case of } F_L > 1.0\text{)} \end{cases}$

Where;$F_{sd}(\Delta u)$; Minimum safety factor that takes into account excess pore water pressure

C; cohesion of soil (kN/m^2)

$\emptyset$; Angle of internal friction

W; Weight of slice (kN)

l ; Length of slice circle (m)

b; Width of slice (m)

u; Pore water pressure usually generated by ground water (kN /m^2)

α; The angle of the normal and the vertical lines that can be placed in the center of the arc(°)

b. K_h Method

○ $F_{sd}(K_h) = \dfrac{\Sigma[c \cdot l + \{(W - u \cdot b) \cdot \cos\alpha - K_h \cdot W \sin\alpha\} \cdot \tan\emptyset]}{\Sigma[W \cdot \sin\alpha + K_h \cdot W \cdot (y/r)]}$

Where K_h ; The design seismic intensity due to the inertia force

y; the height from center of gravity of slice to center of sliding circle (m)

r; Radius of sliding circle (m)

The other symbols are the same used in the equation 1).

Table 4 Relation between Subsidence Amount (maximum) and F_{sd}

$F_{sd}(K_h)$	F_{sd}(Δu)	Subsidence Amount (maximum)
1.0< F_{sd} (K_h)	1.0 < F_{sd} (Δu)	0
0.8 < $F_{sd}(K_h)$ ≦ 1.0	0.8 < F_{sd} (Δu)≦1.0	Embankment height ×0.25
F_{sd} (K_h) ≦ 0.8	0.6 < F_{sd} (Δu)≦0.8	Embankment height × 0.50
—	F_{sd} (Δu)<0.6	Embankment height ×0.75

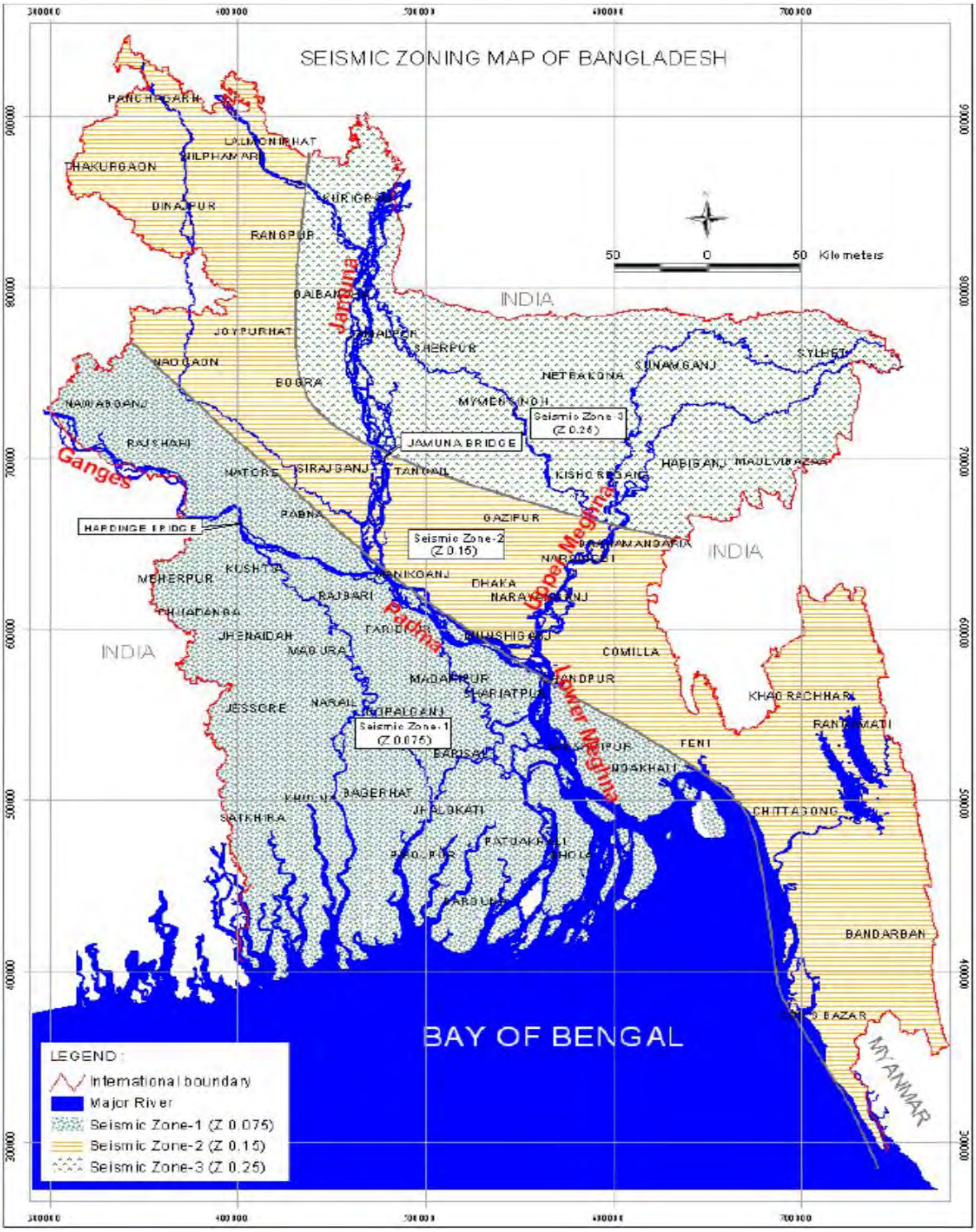


Fig. 34 Seismic Zone Map of Bangladesh (Source; BNBC 1993)

4.4.6 Strengthening work method

The basic anti-seismic method of embankments is to limit or prevent liquefaction in the foundation ground. In principle, the following two methods are available:

1) Increase effective stress by reducing the excessive pore water pressure that occurs during earthquake.
   a. Counterweight filling (making the embankment slopes gentler or providing suppress embankment)
   b. Compacting the foundation ground
   c. Solidifying ground (by cement, burnt lime etc.)

2) Limit ground soil movement to the sides of liquefied ground.
   ・Steel piles are driven in the ground at both sides of embankment toe

Moreover, in the massive earthquake that struck the west part of Honshu island of Japan in January 1995 with magnitude of 7.3 and maximum acceleration of 848 gal, although not directly intended to quell liquefaction, artificially created set-back and gentle embankment slopes became a form of counterweight filling and helped stabilize embankments, thereby demonstrating the effectiveness of method 2) above.

In Bangladesh, since embankments are not very high, it is deemed advisable to adopt rational methods (counterweight filling, making embankment slopes gentler, etc.) that do not incur large investment cost.

4.5 Verification of Embankment Safety with respect to bearing capacity of foundation ground

4.5.1 General

Construction of an embankment on a soft ground faces following geotechnical problems if the soft ground has its inherent characteristics of causing these problems.

A. Excessively large amount subsidence of the embankment due to the high compressibility of the soft ground.
B. Sliding failure of the embankment due to insufficient bearing capacity of the soft ground.

Among these two, the problem of A is dealt with by extra banking in the designing stage to accommodate the future subsidence to keep the required height for securing the function of the embankment. (Please refer 2.6.4)

On the other hand, the problem B occurs due to insufficient bearing capacity of the soft ground. Therefore, in this section, method to deal with the problem B is described.

4.5.2 Process of sliding failure due to insufficient bearing capacity of a soft ground

1) When the soft ground is predominantly clayey silt

Both a deformation due to volume reduction of the ground along with consolidation and a deformation due to shearing stress acting on the ground induced by construction of an embankment likely occur when embankment are constructed on a soft ground.

In other words, with rising of the embankment height, the ground under the embankment subsides and at the same time parts of ground soil pushed out laterally end up heaving upward of the ground surface adjacent to the embankment.

In this situation, if the vertical load on the ground imposed by the embankment exceeds critical bearing capacity of the ground (critical embankment height: HEC), the shearing stress induced by construction of the embankment exceeds the shear strength of the ground leading to sliding failure of the embankment.

Safety factor of slope stability of an embankment (F_S) decreases with the increase of shear stress acting on the ground during the construction of the embankment. This safety factor (F_S) will turn to be on gradual increase, once the construction of an embankment is finished with the increase of shearing strength along with the progress of consolidation.

However, if a period of construction of an embankment is very short, the shearing stress acting on the ground will increase rapidly before the shear strength start to increase leading to abrupt sliding failure of the embankment.

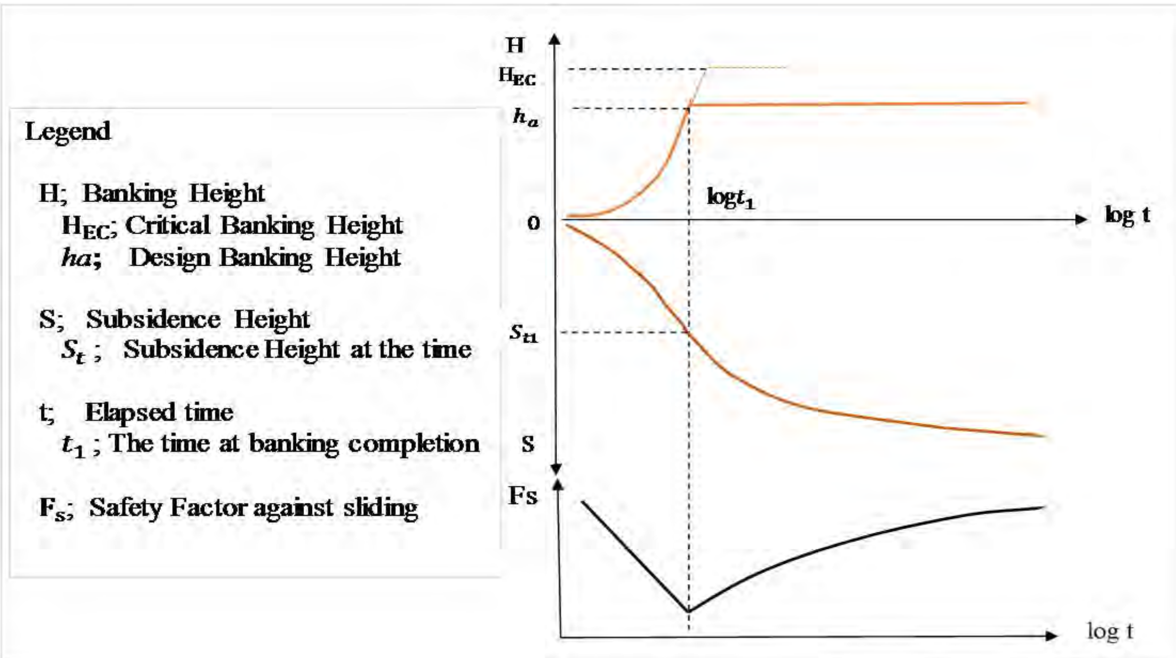


Fig. 35 Variation of Safety Factor of Slope Stability during Consolidation

2) When the soft ground is predominantly sand

A deformation is triggered only by the shearing stress acting on the ground induced by construction of an embankment.

As increase of shear strength by consolidation cannot be expected, sliding failure occurs when shearing stress exceeds initial shear strength of the ground.

It is noted that safety factor should be 1.2 or higher in these cases.

4.5.3 Soil constants for circular slide calculation of soft ground

Stability of embankment on soft ground is assessed based on minimum safety factor by circular slip calculation on assumed circular slip surface in the ground.
Calculation method is described in 4.2.3 Circular Slip, soil constants for calculation are as follows.

1) In the case that the ground is predominantly clayey silt

C and $\emptyset$ for un-drained condition should be identified because of its low permeability. These C and $\emptyset$ are expressed as Cu and $\emptyset_u$ respectively.

Un-drained Cu of each slice along the potential sliding slip of the slope stability analysis is determined considering increase by the depth of the slice from the ground surface and increase by the consolidation. $\emptyset_u$ is assumed to be zero (0.0).

Here, Cu is obtained from CU tests (Consolidated Un-drained Tri-axial Compression Tests) in which different confining pressures are applied to samples taken from same depth.

Cu can also be obtained from UU tests (Un-consolidated Un-drained Tri-axial Compression Tests) in which different inherent confining pressures are applied to each sample from different depths.

In the case that Cu is obtained from the following equation based on CU tests.

$$Cu = C_{cu} + P \cdot \tan(\emptyset_{cu})$$

Here P: Effective overburden pressure acting on each slice along a potential sliding slip after embankment. P is expressed as ($P = P_0 + \Delta P$) (kN/m^2)

P_0: Effective overburden pressure before embankment (kN/m^2)

ΔP : Added effective overburden pressure induced after embankment (kN/m^2)

C_{cu}: Cu value at P =0.0 on a envelop line of Mohr's Circles expressed with total stress (kN/m^2)

$\emptyset_{cu}$: Gradient of the envelop line representing increase of Cu (°)

When the effective overburden pressure acting on each slice along the potential sliding slip before embankment is expressed as P_0 (kN/m2) and the added effective overburden pressure on the slice induced by embankment is expressed as ΔP (kN/m^2), P = P_0+ ΔP is used for the equation.

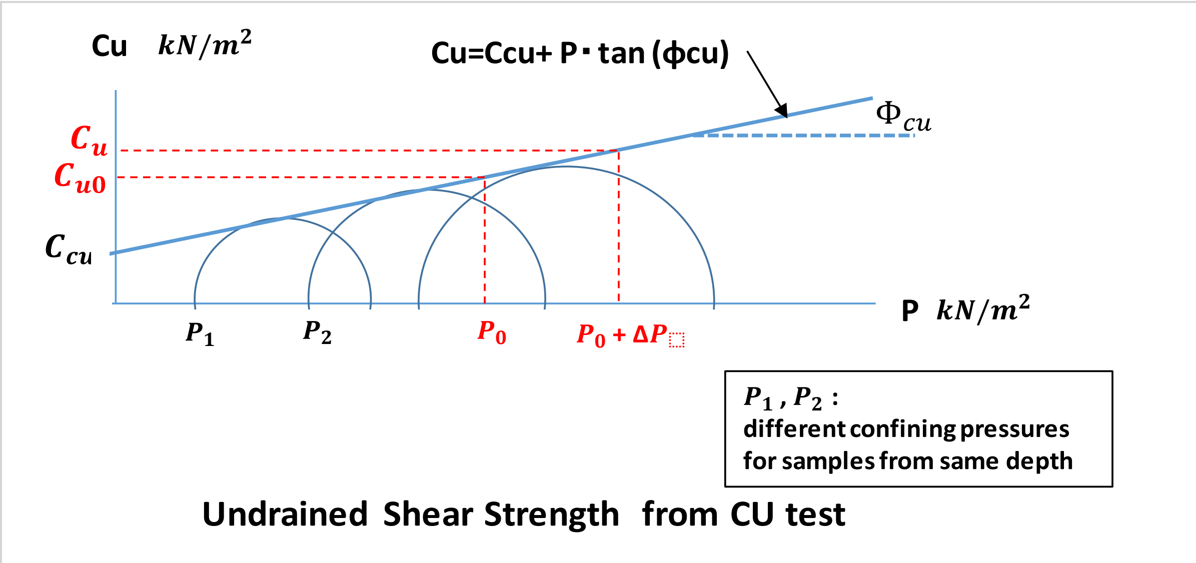


Fig. 36 Shear Strength of Clayey Silt (CU tests)

2) In the case that the ground is predominantly sand

C and ∅ for drained condition should be identified because of its high permeability. These C and ∅ are obtained from CU tests by eliminating the effect of pore water pressure generating during un-drained condition of CU tests.

4.5.4 Countermeasures for stability of embankment constructed on soft ground

(1) Counterweight Banking Method or Slow Loading Method

Counterweight Banking Method aims to increase resistance moment by putting extra banking on both sides of embankment or making slope gentle against sliding of embankment.

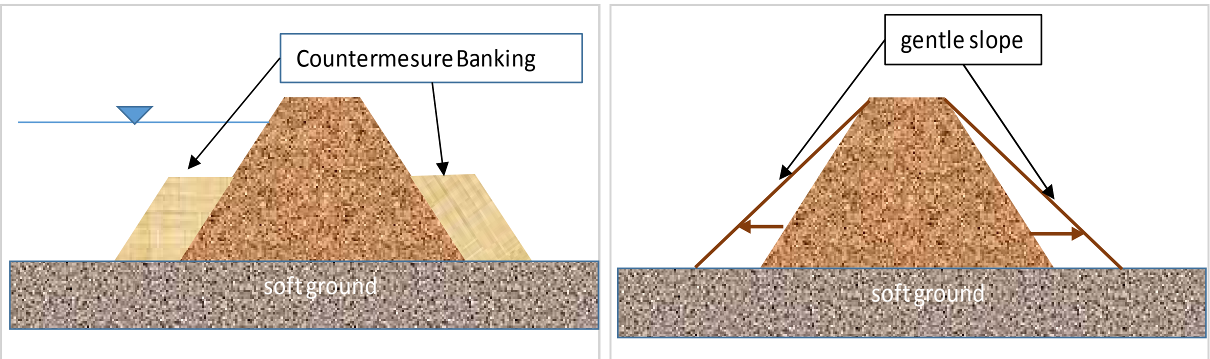


Fig. 37 Counter Weight Banking Method

Method is intended to bank counterweight soil slowly. Increment of shearing force can be expected along with progress of consolidation due to loading of soil in the case of slow loading on soft clayey ground.

(2) Mitigation of Liquefaction Hazard by Site Modification

1) Excavation and replacement of liquefiable soils

a. Excavation and engineered compaction of the existing soil

b. Excavation and engineered compaction of the existing soil with additives

c. Excavation of existing soils and replacement with properly compacted non liquefiable soils

2) Densification of in-situ soil

a. Compaction piles

b. Vibratory probes

c. Vibroflotation

d. Compaction grouting

e. Dynamic compaction or impact densification

3) In-situ improvements of soil by alteration

a. Mixing soils in-situ with additives

b. Removing in-situ soils by jetting and replacement with non-liquefiable soils

4) Grouting or chemical stabilization

THE PROJECT FOR
CAPACITIES DEVELOPMENT OF MANAGEMENT FOR SUSTAINABLE WATER RELATED INFRASTRUCTURE
IN
THE PEOPLE'S REPUBLIC OF BANGLADESH

PILOT PROJECT DESIGN

REPORT

May 2015

BWDB Design Circle-1
JICA Expert Team

Table of Contents

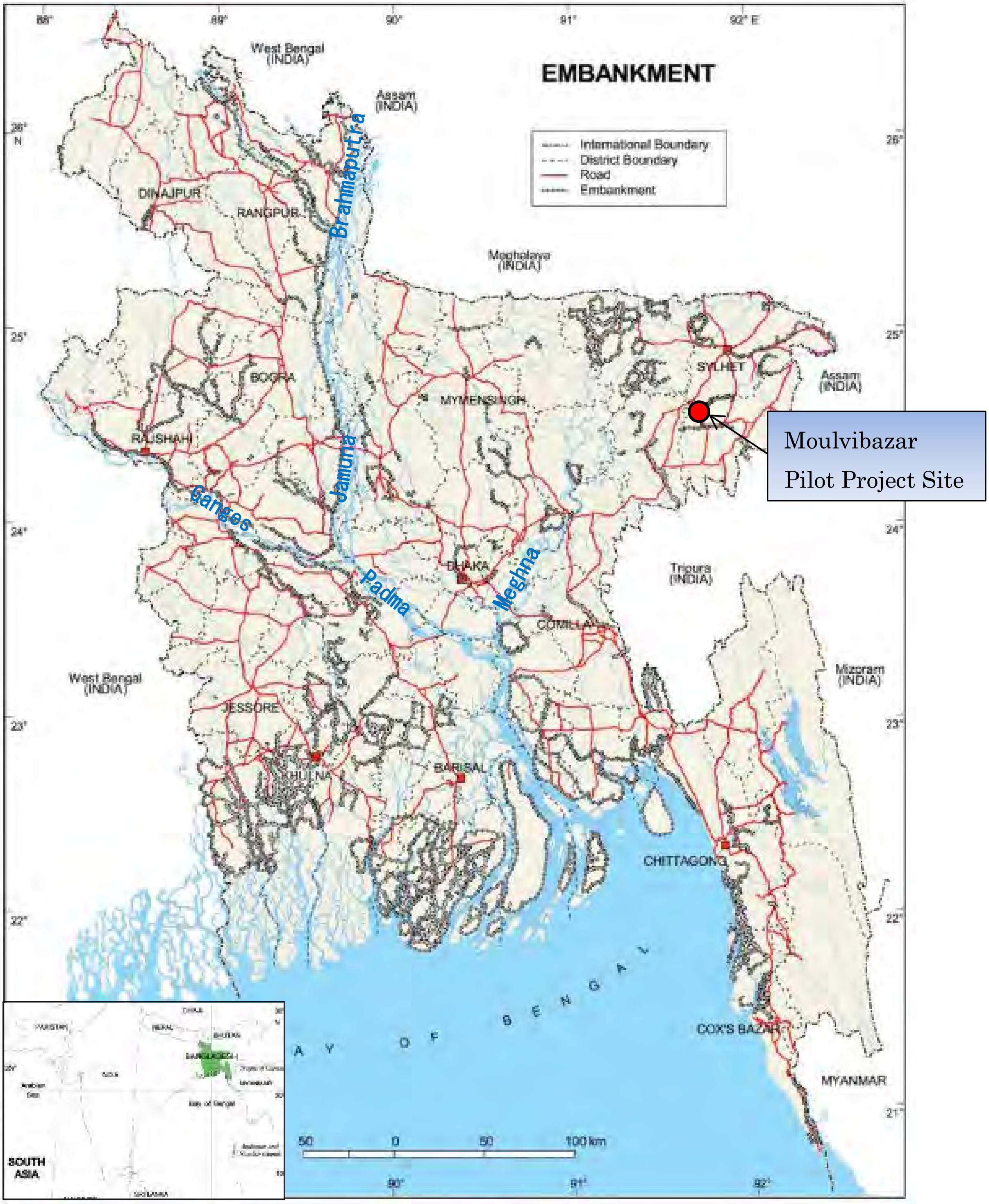


Figure 1 General Location of Pilot Project

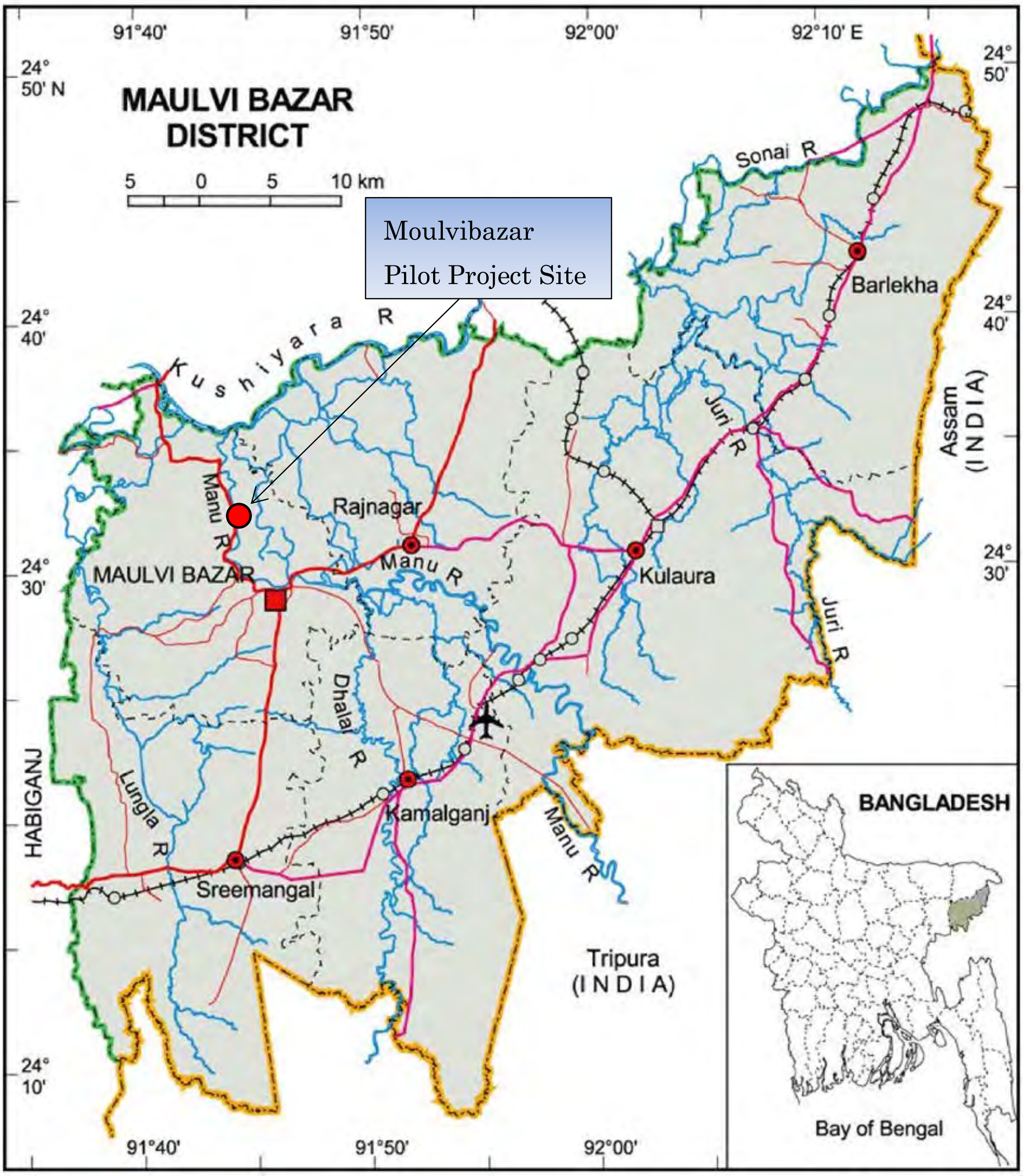


Figure 2 Location of Manu River

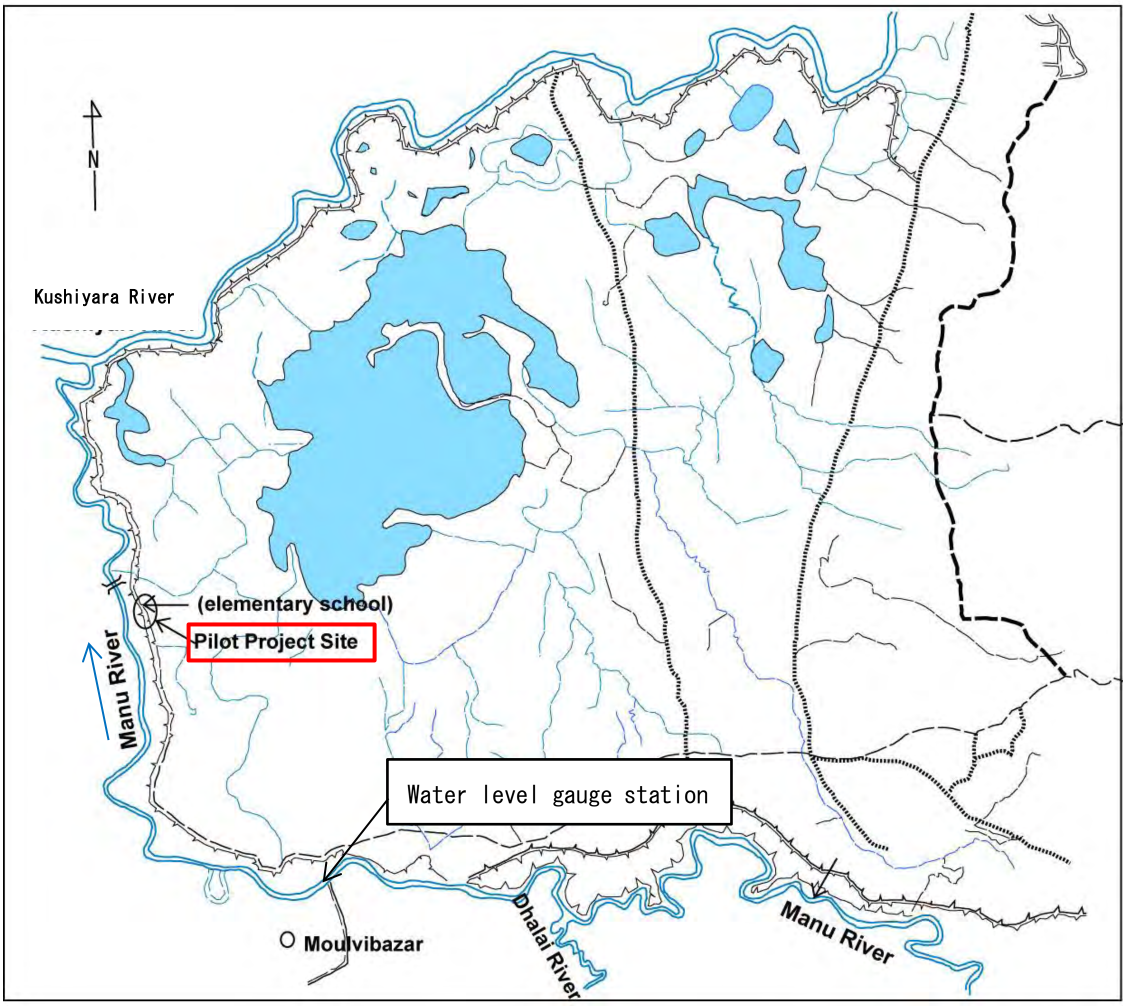


Figure 3 Detailed Location of the Pilot Project

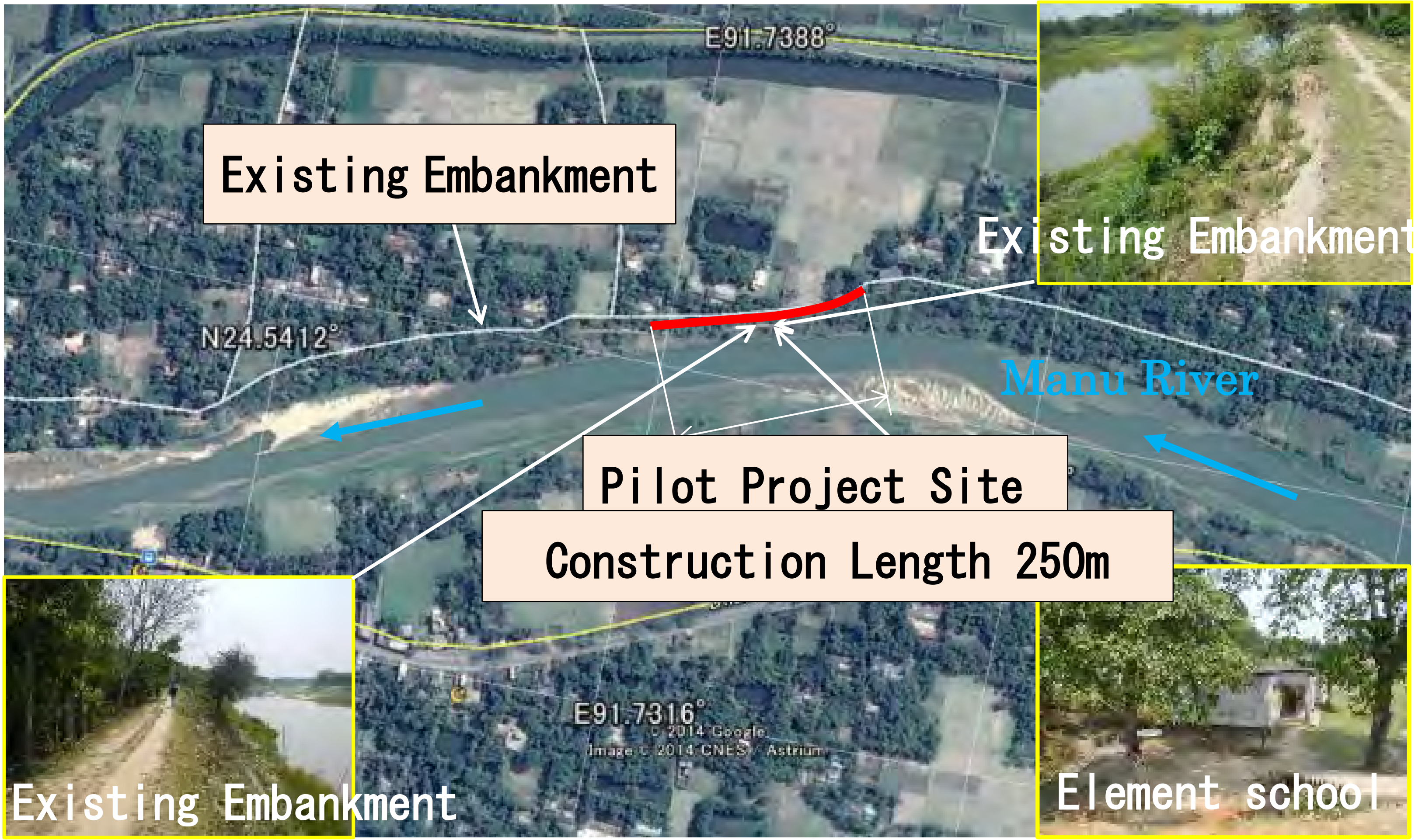


Figure 4 Pilot Project Site

1. Pilot Project

1.1 Purposes of the Pilot Project

"The Project for Capacity Development of Management for Sustainable Water Related Infrastructure" (hereinafter referred as "the Project") is being carried out with the following expected goal and output:

(1) Overall Goal

To achieve water-related disaster risk reduction through proper management of the infrastructures

(2) Goal of the Project

To improve the capacities of BWDB on Embankment Engineering in terms of Design, Construction and Operation & Maintenance methods

(3) Outputs

a. Design for sustainable river embankment is introduced.
b. Construction method and procedure of river embankment is improved.
c. Operation and Maintenance (hereinafter referred to as "O&M") system for the river infrastructures is ensured

Within the framework of the Project, a pilot project consisting of reconstruction of a section of existing embankment is being conducted with the following purposes by the cooperation of BWDB and the JICA Expert Team as the basic item of on-the-job training (OJT):

a. To verify effectiveness of the draft manuals of design, construction and O&M to be developed under the Project
b. To revise the draft manuals through the monitoring of the pilot project

This report of the detailed design is prepared for the Pilot Project construction in order to prepare the required drawings and quantities in collaboration with the BWDB design Circle-1.

1.2. Pilot Project Target Site

Out of 21 sites initially recommended by BWDB, field surveys at 13 sites were conducted.

Further, as a result of conducting the phases evaluation described below, the damaged embankment at Moulvibazar was selected as the Pilot Project site (see Table 1). Figures 1~4 show the location of the Pilot Project site.

Evaluation items in the 1st evaluation

① Large-scale erosion protection is not required on the site. (Because the project is mainly intended to improve embankment design and construction works)

②Civil engineering technology is widely applicable to Bangladesh.

③ There is a high level of urgency for embankment construction and/or rehabilitation.

④ There is no risk of the embankment damaged by artificial activity such as public cutting.

Evaluation items in the 2nd evaluation

① Evaluate of workability (ease of procuring and carrying-in equipment and materials)

② Social and environmental consideration (there is no need to acquire land or move houses, etc.)

③ Access from Dhaka

Table 1 Pilot Project Site Selection Sheet

Category		No.	Site Name	Elevation (water surface level)	River				Embankment					1. Suitability Evaluation in Implementation						2. Ease in Execution				Overall Evaluation
					River	Width of River	Bed Gradient		Type of Embankment	Level of Embankment (estimated)	Embankment Soil Quality (estimated)	Current Banking Method	Cause of Embankment Failure (estimated)	① Need or Not for Erosion Countermeasures	② Technology Transfer	③ Urgency of Embankment Improvement	④ Embankment Damage Caused by Human Activity	Suitability Evaluation		⑥ Ease of Equipment Procurement and Carrying-in	⑦ Social and Environmental Consideration (Land, etc.)	⑧Access from Dhaka	⑨ Overlap with Other Agencies	
A		1	Bogra	15m	Jamuna River	7km	1/10,000	HWL is for a long time	Full-embankment (Jamuna River embankment)	Approx. 3m	Sandy silt	Dredged sediment (fine sand)＋Clay covering (30cm)	Bank erosion	C Erosion countermeasures needed	Durable clay covering	C (Priority to erosion countermeasures)	None	X	Erosion countermeasures needed				ADB	
	Large river	2	Sirajiganj	14m	Jamuna River	13km	1/25,000	HWL is for a long time	Full-embankment (Jamna River embankment)	Approx. 2m	Fine sand	① Geo-textile, CC blocks ② Clay covering	Bank erosion	C Erosion countermeasures needed	Durable clay covering	C (Priority to erosion countermeasures)	None	X	Erosion countermeasures needed				ADB	
	embankment	3	Rajshahi (1)	12m	Ganges River	1km	1/4,000	HWL is for a long time	Full-embankment (Ganges River embankment)	Approx. 2m	Clayey silt	① Geo-textile、CC blocks ② Clay covering (30cm)	Bank erosion	C Erosion countermeasures needed	Durable clay covering	C (Priority to erosion countermeasures)	None	X	Erosion countermeasures needed					
		4	Chandpur	0m	Padma River	2.8km	1/10,000	HWL is for a long time	Full-embankment (Padma River)	Main road-cum-embankment (Approx. 3m)	Silt	Geo-textile, CC .blocks	Bank erosion	C Erosion countermeasures needed	Durable clay covering	C (Priority to erosion countermeasures)	None	X	Erosion countermeasures needed				ADB	
B	High tide area embankment	5	Khulna	2m	Ganges River (branch)	400m	1/50,000	HWL is for a long time Tidal variations	Full-embankment (Polder embankment)	Approx. 5m	Clayey-silt (with peat)	Manual compaction by rammer	① Bank erosion and waves ② Partial public cut ③ Seepage failure with peat	B (Fast erosion)	X	A	None	△	Transfer technology?	△	○	△	△WB	2
C	Haor	6	Habiganj (2)	8m	Khowai River	130m	1/100,000	Prolonged inundation	Submerged embankment	Approx. 3~5m	Sandy-silt	Manual compaction by rammer	Overflow	A Unnecessary	Submerged embankment	B	None	X	Submerged embankment					
	embankment	7	Nerokona	8m		125m	1/100,000	Prolonged inundation	Submerged embankment	Approx. 3m	Clayey-silt	Manual compaction by rammer	Public-cut, overflow, crab holes	A Unnecessary	Submerged embankment	B	※Yes	X	Submerged embankment					
D		8	Feni	25m	Muhuri River	15m	1/60	Rapid rises and falls	Full-embankment (Flash-flood)	Approx. 5m	Clayey silt	Unknown	Foot scouring (flash flood)	B Foot protection	Compaction and foot protection	C (Small beneficiary area)	None	X	Urgency None					
		9	comilla	13m	Gumiti River	40m	1/100,000	Rapid rises and falls	Full-embankment (Flash-flood)	Approx. 5m	Sandy-silt	CC blocks	① Piping, ② Rat holes, ③ Foot scouring (flash flood)	B Foot protection	Compaction and foot protection	C (Done except for foot protection; partial failure)	None	○		△	△	○	None	3
	River embankment with steep slope behind	10	Moulvibazar	16m	Manu River	85m	1/5,000	Rapid rises and falls	Full-embankment (Flash-flood)	Approx. 5m	Sandy-silt	Stone revetment CC blocks	Foot scouring (flash flood)	B Foot protection	Compaction and foot protection	A	None	○		○	○	○	None	①
		11	Habiganj (1)	23m	Khowai River	60m	1/1,000	Rapid rises and falls	Full-embankment	Approx. 5m	Silt	Unknown	Foot scouring (flash flood) Artificial crest damage	B Foot protection	Compaction	B	○	X	Artificial causes					
		12	Rajshahi (2)	15m		100m	1/100,000	Unknown	(Polder embankment) Full-embankment	Approx. 3m	Clayey silt	Unknown	Water pressure? Rate holes? Public cut?	A Unnecessary	Compaction and foot protection	C	※Yes	X	Public cut					
E	Others	13	Mymengshigh	12m	Old Bramaputra River	133m	1/100,000	Unknown	Full-embankment	Almost none (reclaimed land to rear)	Sandy silt	Clay covering	① Bank erosion ② Slip caused by rainfall ③ Overflow (△)	C Erosion countermeasures needed	—	C	None	X	Erosion countermeasures needed					

1.3 Design target Facilities

Local scouring is main factor of the existing embankment failure at the Pilot Project site. Therefore, the following structures against local scouring and erosion are designed and integrated into the embankment body:

- Foot protection works to prevent local scouring
- Implementation of slope protection work on outer bank of bend using slope blocks with projections

1.4 Applicable Standards

Design for the Pilot Project is prepared based on the River Embankment Design Manual in Bangladesh (Draft), and the following two standards were also referred to.

- Guideline for River Bank Protection 2010, BWDB
- Dynamically Design Method on Revetment, 2007, Japan Institute of Country-ology and Engineering

Hereafter in this report, the Guideline for River Bank Protection 2010, BWDB is referred to as the Guideline: the Dynamically Design Method on Revetment, 2007, Japan Institute of Country-ology and Engineering is referred to as the Dynamically Design Method on Revetment, and the River Embankment Design Manual in Bangladesh is referred to as the Design Manual.

2. Design Conditions

2.1 Natural Conditions

The pilot site is situated on the outer bank (right-side bank) of a gentle bend in Manu River. The embankment has no revetment work, so riverbed scouring at times of flooding during the rainy season triggers bank erosion and collapse, and there is concern over the impact of this on the elementary school (Primary) and community in the hinterland.

The following sections describe the river and soil quality conditions that provide the basis for design of the embankment and embankment protection works.

2.1.1 River Conditions

(1) Water Level Data at Moulvibazar Water Level Gauging Station

Moulvibazar water level gauging station is located at 10.2km upstream from the pilot project site, and is the nearest water level gauging station to that, as shown in Figure-3. Daily water level of the Manu River has been observed visually every day.

Based on the data from the Processing and Flood Forecasting Circle, BWDB, annual maximum and minimum water levels from 1981 through 2013 at Moulvibazar station are listed in Table-2, and presented in Figure-5.

Also, Figure 5 shows the hydrograph (daily water level) for the period from January 1, 2004 to March 31, 2014. As the annual trend, the water level rises from April and remains high until October. The water level usually starts falling in November and remains low until March, although the start of decline was delayed in 2011.

Accordingly, it will be necessary to complete construction works on the river side of the site by the end of March.

Table 2 Annual Maximum / Minimum Water Levels

StationID	Year	Max WL (mPWD)	Min WL (mPWD)
SW202	1981	11.79	6.19
SW202	1982	12.20	6.07
SW202	1983	12.74	5.77
SW202	1984	13.10	5.51
SW202	1985	12.95	5.60
SW202	1986	12.46	5.59
SW202	1987	12.11	5.50
SW202	1988	12.95	5.34
SW202	1989	12.41	5.70
SW202	1990	11.96	5.64
SW202	1991	12.90	5.74
SW202	1992	10.75	5.77
SW202	1993	13.19	5.82
SW202	1994	10.90	5.65
SW202	1995	12.65	5.59
SW202	1996	11.97	5.58
SW202	1997	12.55	5.56
SW202	1998	11.68	5.66
SW202	1999	11.22	5.64
SW202	2000	12.61	5.78
SW202	2001	12.87	5.80
SW202	2002	12.50	6.21
SW202	2003	12.80	6.17
SW202	2004	13.15	6.22
SW202	2005	12.78	6.17
SW202	2006	12.13	6.04
SW202	2007	12.68	6.41
SW202	2008	10.90	5.71
SW202	2009	11.04	5.89
SW202	2010	12.22	5.76
SW202	2011	11.41	5.46
SW202	2012	12.08	5.95
SW202	2013	11.63	5.94

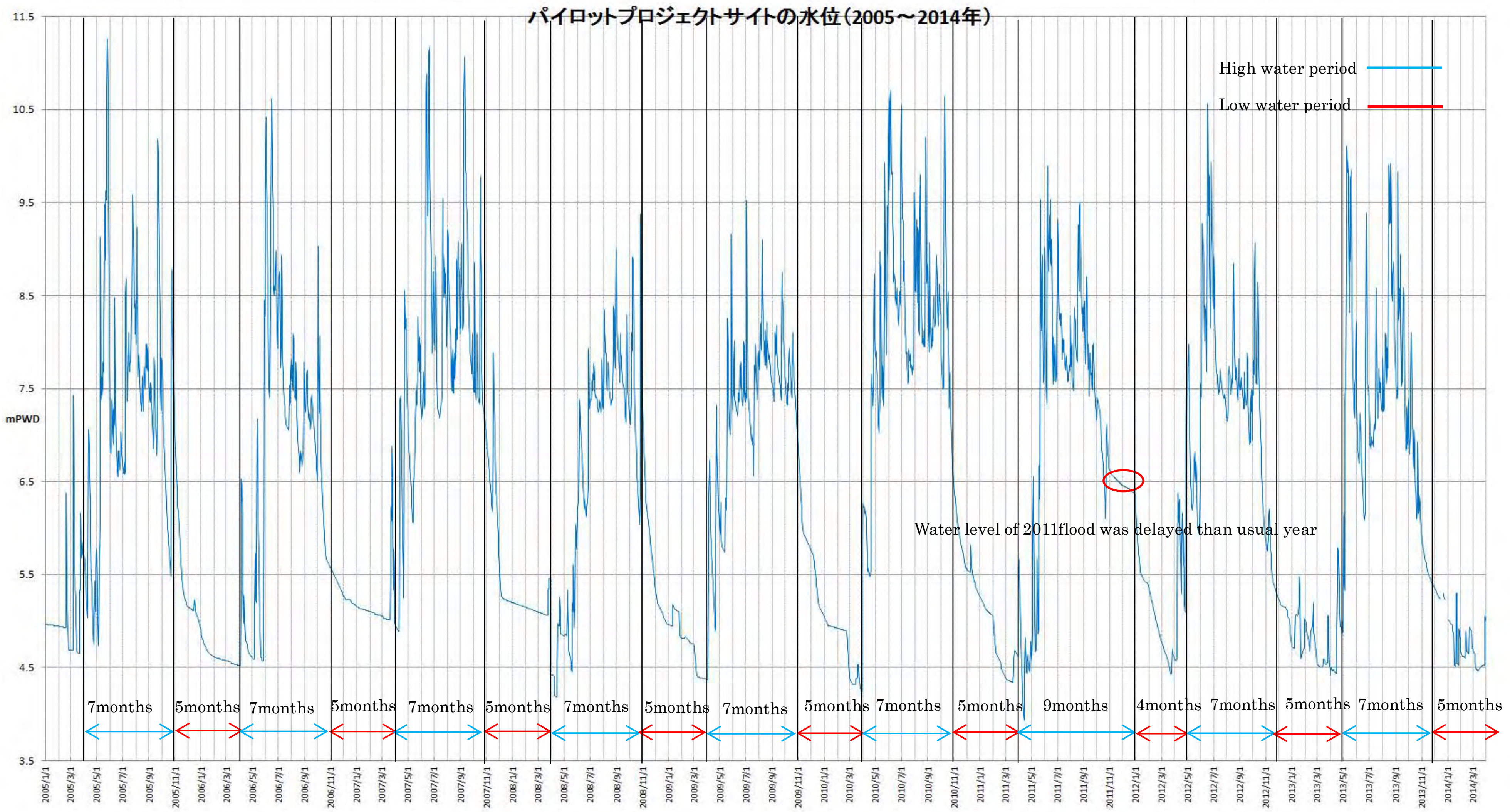


Figure 5 Pilot Site Hydrograph (January 1, 2005~March 31, 2014)

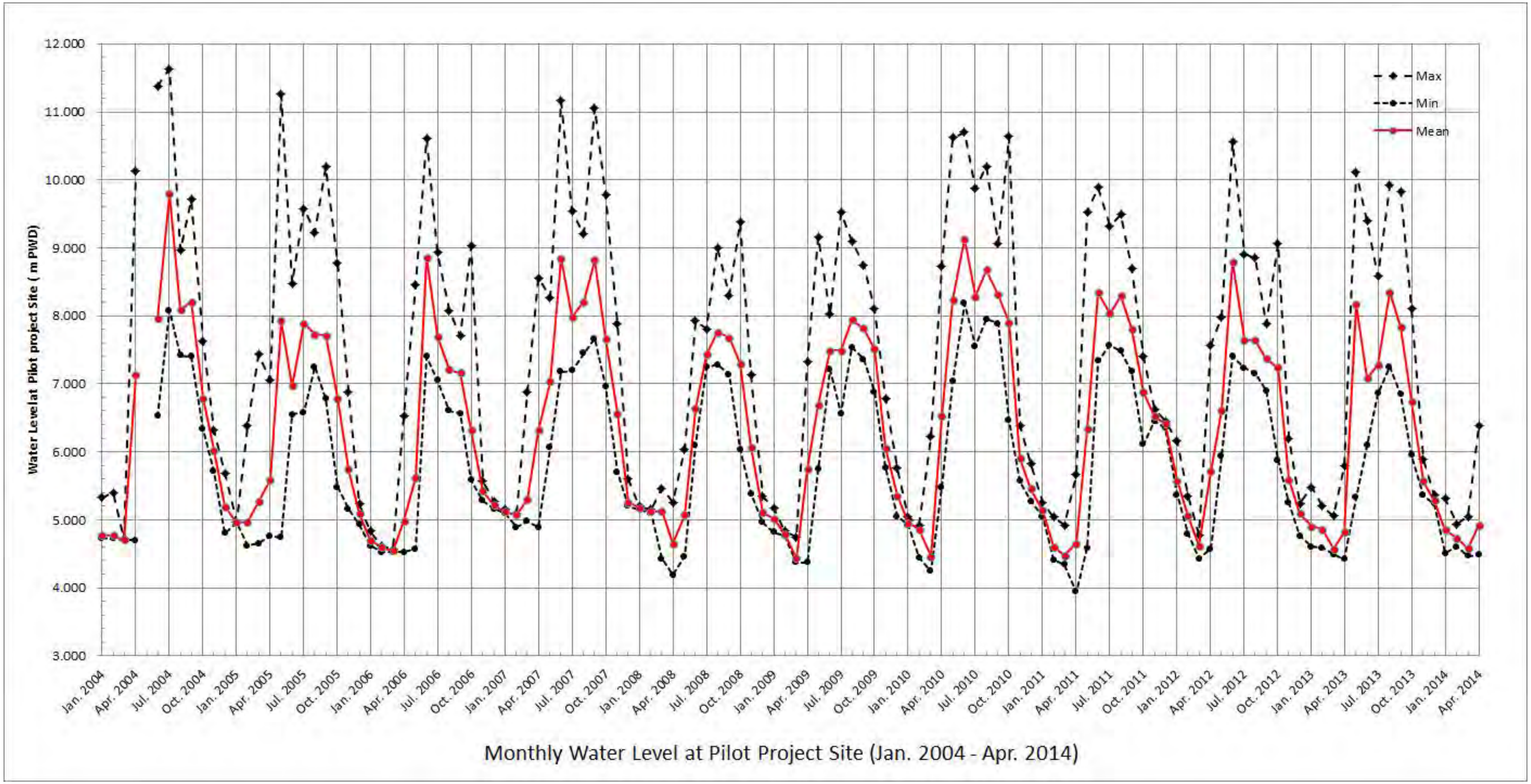


Figure 6 Pilot Site Fluctuations in Monthly Water Level (Maximum, Minimum, Mean Water Level)

(2) Riverbed Fluctuations at Moulvibazar Water Level Gauging Station

The annual maximum water levels fluctuate between 11 m PWD and 13 m PWD. Unilateral fluctuation trends cannot be observed from the data and extremely decrease and rise of river level is not confirmed

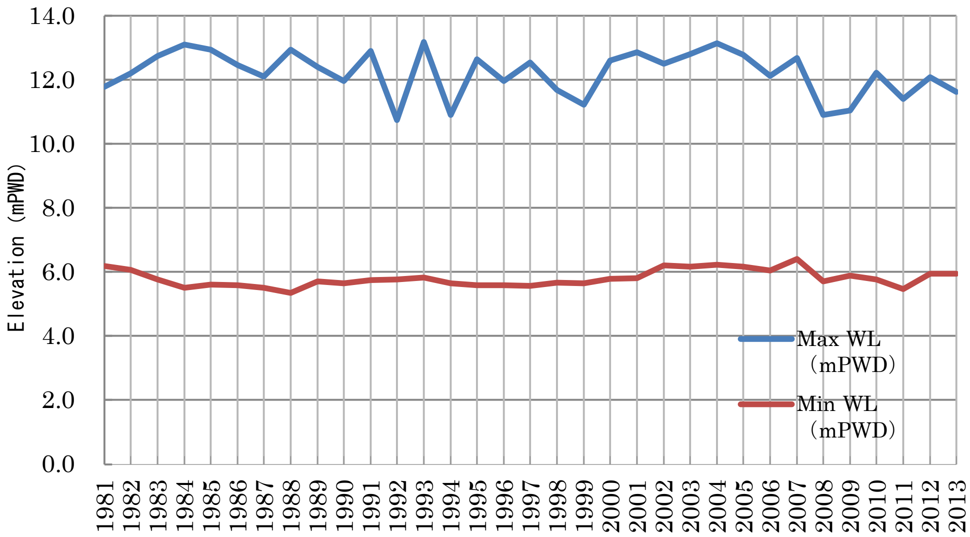


Figure 7 Annual Maximum and Minimum Water Level
(Moulvibazar Water Level Gauging Station)

Cross sectional profiles at Moulvibazar Station surveyed at 2002 and 2005 are shown in Figure -8. According to the figures, there is no remarkable river bed fluctuation observed at the Moulvibazar Station.

As there is no remarkable fluctuation of the annual water level data and the river bed of the river channel around the Moulvibazar Station, it is judged that the water level data of Moulvibazar Station are reliable for the probability analysis of the water levels.

(Reference)

According to the information from the staff in charge of the observation, datum elevation of the staff gauge is set at 12.140 m PWD, which is situated near the bridge and surveyed from the B.M. maintained by the neighboring BWDB office.

The staff gauge is installed on the pier of the bridge across the Manu River. The staff has observed the water level visually and recorded three times a day. The record book in which data of several months is compiled is sent to the Processing and Flood Forecasting Circle in Dhaka.

Photographs of Moulvibazar Station are shown in Photo-3 below.

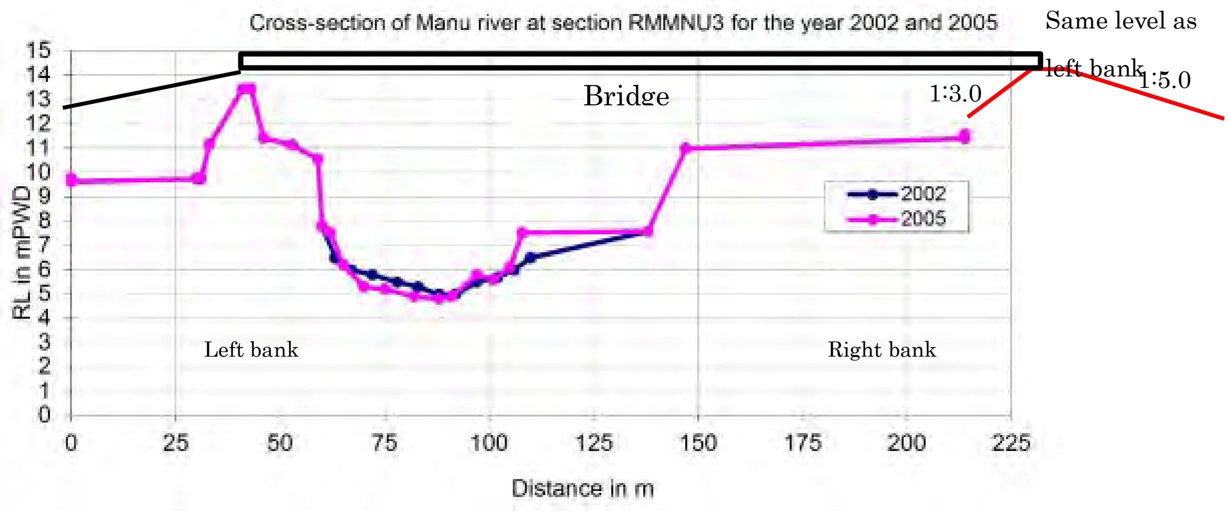


Figure 8 Cross Section of Rriver at Moulvibazar Water Level Gauging Station

Table 3 Moulvibazar Water Level Gauging Station Photographs

 Overview of the observation point	 BM used by the station
 Water level staff gauge (pier of bridge)	 Gauging Station: Elevation 12.140m PWD

	
Right bank	Gauge reader of station (right side)

(3) Probable Water Level at Moulvibazar Water Level Gauging Station

There is no established standard method for the probability analysis of hydrological data in Bangladesh. Therefore, the method of probability analysis of hydrological data applied in Japan is introduced and applied to the probability analysis of water levels in Moulvibazar Station as a reference.

The probability distribution methods applied in Japan are shown in below Table:

Probability Distribution Method applied in Japan

Log-normal distribution	Two param of the log-normal distribution method (LN)
	Three param of the log-normal distribution method (LN3PM)
	Three param of the log-normal distribution quintile method(LN3Q)
	Ishihara-Takase method
	Iwai method
	Log-normal L distribution method(LNLM)
Log Pearson III type distribution	Log Pearson III type distribution method(LogP3)
	Log Pearson III type distribution method (LogP3)
Exponential distribution	Exponential distribution (EXP)
Extreme value distribution	Gumbel method
	SQRT-exponential type method (SQRT-ET)
	Generalized extreme value method (GEV)

The probable hydrological values are estimated through the following process:

i) Screening of probability distribution methods by goodness-of-fit evaluation:
For the objective hydrological data, the Standard Least-Squares Criterion (SLSC) of each probability distribution method is examined. The methods with the SLSC value less than 0.04 are screened.

ii) Selection of appropriate probability distribution method by stability evaluation:
Among the screened methods by SLSC examination, the probability values and estimate errors of the screened methods are computed by use of the Jackknife resampling method that statistically corrects the bias, in order to evaluate the stability of each method and to finalize the most appropriate method. The method with minimum Jackknife's estimate error is selected for the probability analysis of the objective hydrological data.

The probable water levels at Moulvibazar Station are calculated by the maximum recorded water level data as shown in Table-2, and the estimation results are as follows:

<u>Goodness-of-fit evaluation of probability distribution method：</u>
Gev, LP3Rs, and LogP3 methods are screened with adaptability.

<u>The sustainability evaluation of the probability distribution method (Finalize the selection of the method)：</u>
GEV method is finally selected as the probability distribution method for analysis due to suggestion of minimum bias as the result of Jackknife estimate method.

<u>The identification of the probability water levels：</u>
Jackknife estimate values as the probable water levels at Moulvibazar Station is calculated by the GEV and are shown in Table – 4.

Table 4 Probable Water Levels at Moulvibazar Station (GEV Method)

Return Period (year)	Estimated water level: Jackknife estimate (m PWD)	Error range (Jackknife estimation error) (m)
2	12.35	0.15
3	12.64	0.12
5	12.87	0.10
10	13.06	0.08
20	13.17	0.09
30	13.21	0.10
50	13.25	0.12
80	13.27	0.13
100	13.28	0.14
150	13.30	0.14
200	13.31	0.15

(4) Riverbed Gradient of Manu River

According to the pre-existing report (※), which indicates the water surface gradient at the annual mean flood discharge, the water surface gradient of Manu River is 1/6700, while the riverbed gradient is given as 1/6800 based on the longitudinal profile.

(※) Source: Final Report (Main Report), Feasibility Study for A Comprehensive Study for Drainage and Flood Management in the Manu-Dhalai-Kushiyra-Khowai Systems, BWDB (Development Design Consultants Limited in association with Engineering & Planning Consultants Limited), June 2003

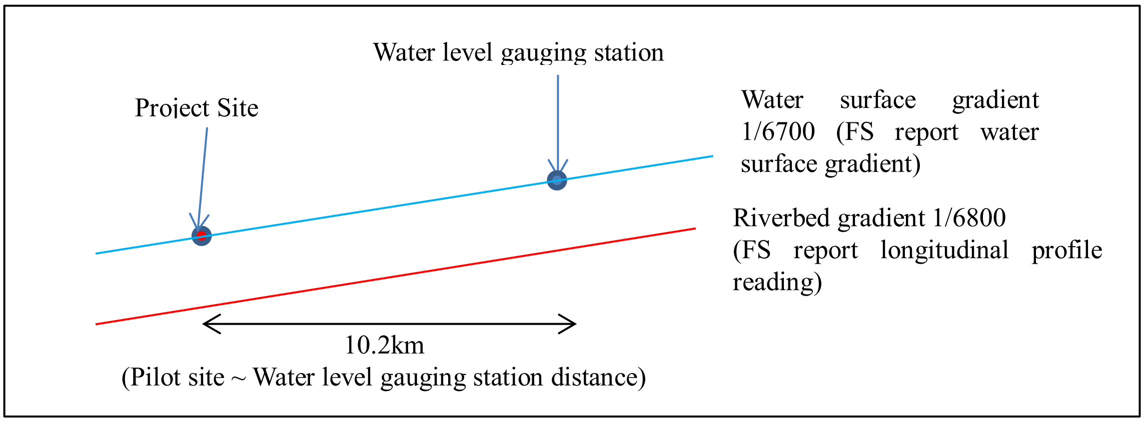


Figure 9 Longitudinal Profile between Pilot Project Site and Water Level Gauging Station

2.1.2 Soil Quality

In order to grasp soil conditions in the local area and collect basic data for verification of embankment stability, boring investigation was implemented at three locations, and soil investigation was implemented at one location that is a potential borrow pit. Figure 10 shows the boring locations and Figure 11 shows the borrow pit candidate site.

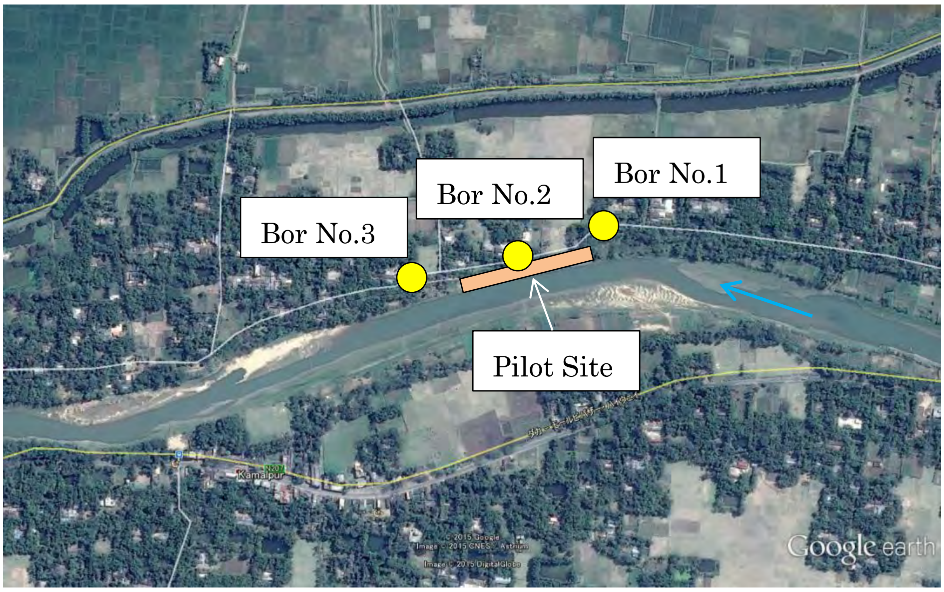


Figure 10 Boring Investigation Locations

(1) Results of Boring

The soil quality in the local area is mainly composed of silt: soft layers with an N value of less than 4 are partially confirmed, and N values at different elevations are similar. The elevations of each stratum are similar across the site, indicating that the area generally has uniform strata.

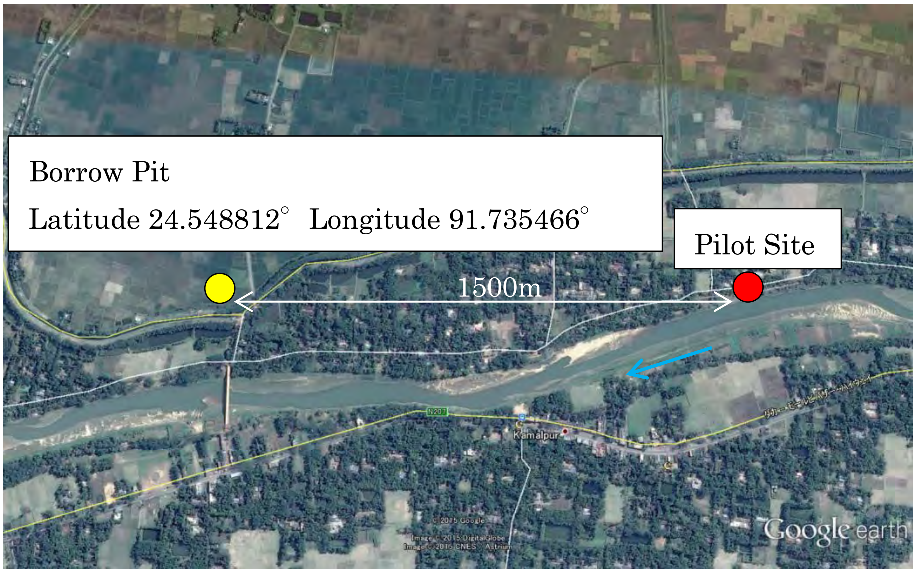


Figure 11 Sampled Location (Candidate Borrow Pit Site)

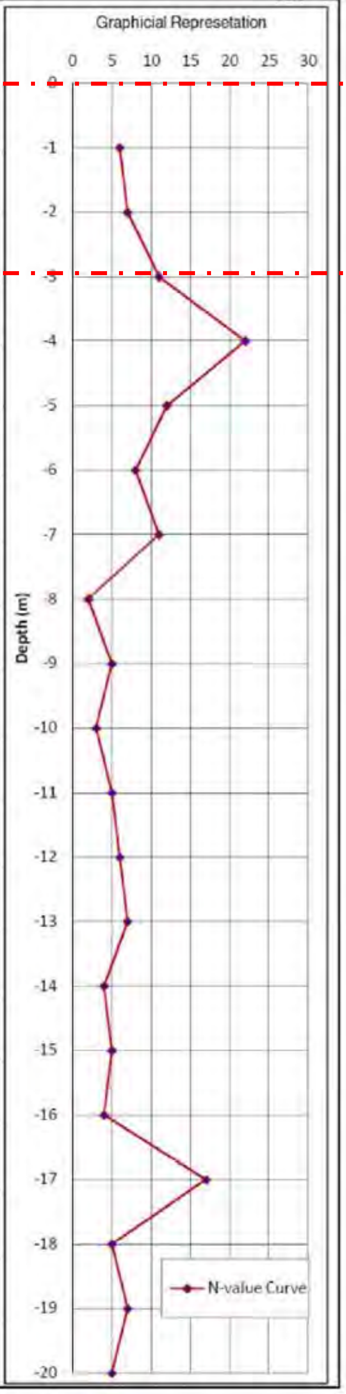
Boring No. 2

Project: Capacity Development of Management for Sustainable Water Related Infrastructure	**Client: JICA**
	Date :19-04-2014
Borehole No : **BH-02**	Geographic Coordinates (WGS 84): Lat- 24.537109°, Long- 91.735708°
Location: Noarai Primary School, Akhailkura, Moulavibazar	Depth of Borehole (m): 20.00
Borehole Diameter (cm): 10.00	Ground Water Level (m) : 6.80
Boring Method: Machine wash boring	Ground Elevation (m): 12.074
Type of Bit: Chisel	Legend: D Disturbed Sample; UD Undisturbed Sample; Sand; Clay; Silt; N-Value; Organic Clay
Type of Hammer: Auto Donut Hammer	

Depth Below G.L. (m)	Thickness (m)	Lithologic Description
0.0–2.0	2.0	Yellowish brown medium stiff fine sandy **SILT** trace clay
2.0–3.0	1.0	Yellowish grey to brown grey stiff clayey **SILT** with sand
3.0–4.0	1.0	Yellowish grey to brown very stiff sandy **SILT** with clay
4.0–5.0	1.0	Browish grey stiff sandy **SILT** little clay
5.0–6.0	1.0	Bluish grey medium stiff silty **SAND** trace clay
6.0–7.0	1.0	Bluish grey stiff clayey **SILT** trace sand and organic materials
7.0–8.0	1.0	Bluish grey very soft clayey **SILT** trace sand
8.0–9.0	1.0	Bluish grey medium stiff clayey **SILT** trace sand
9.0–11.0	2.0	Bluish grey to black soft to medium stiff clayey **organic SILT**
11.0–13.0	2.0	Dark grey to black stiff clayey **Organic SILT** and **PEAT**
13.0–16.0	3.0	Bluish grey medium stiff calyey **SILT** trace sand
16.0–17.0	1.0	Bluish grey medium dense silty **SAND** trace clay
17.0–18.0	1.0	Bluish grey medium stiff clayey **SILT** trace snad
18.0–20.0	2.0	Bluish grey medium stiff sandy **SILT** with clay

Sample Depth (m)	Sample No
0.10	UD-1
0.55	D-1
1.10	UD-2
1.55	D-2
2.10	UD-3
2.55	D-3
3.10	UD-4
3.55	D-4
4.10	UD-5
4.55	D-5
5.10	UD-6
5.55	D-6
6.55	D-7
7.10	UD-7
7.55	D-8
8.55	D-9
9.10	UD-8
9.55	D-10
10.55	D-11
11.10	UD-9
11.55	D-12
12.55	D-13
13.10	UD-10
13.55	D-14
14.55	D-15
15.10	UD-11
15.55	D-16
16.55	D-17
17.10	UD-12
17.55	D-18
18.55	D-19
19.10	UD-13
19.55	D-20

SPT Interval (m)	15cm Penetration	15cm Penetration	15cm Penetration	N-Values
1.0	2	3	3	6
2.0	3	3	4	7
3.0	4	5	6	11
4.0	6	10	12	22
5.0	3	5	7	12
6.0	2	4	4	8
7.0	6	6	5	11
8.0	1	1	1	2
9.0	2	2	3	5
10.0	1	1	2	3
11.0	1	2	3	5
12.0	2	3	3	6
13.0	3	3	4	7
14.0	1	2	2	4
15.0	1	2	3	5
16.0	1	2	2	4
17.0	3	7	10	17
18.0	2	2	3	5
19.0	1	3	4	7
20.0	1	2	3	5



Hole level 12.074mPWD

Embankment body

Ground level inside embankment 9.1mPWD

Foundation

(2) Soil Constants for Design

In the Pilot works, because the locally available banking materials are limited to silt and clayey soil, it will be difficult to apply the 90% degree of compaction that is prescribed in Japan, where generally good quality earth is used.

Meanwhile, in BWDB embankment works, 85% compaction is usually adopted, although 90% is adopted in some donor projects.

Since the objective of the Pilot Project is to provide a model of small-medium river embankment improvement that is affordable under the BWDB budget, the degree of embankment compaction has been set at 85%. The following table shows the soil constants used in the design.

As a result of laboratory test, Cohesion value is shown 29.41KN/m^2, but 15kN/m^2 of embankment general cohesion value is adopted in pilot project embankment design, because 29.41KN/m^2 is too huge value for Construction Company.

Table 5 Soil Constants (Shear Strength, Coefficient of Permeability)

	Embankment	Undergroud
Wet unit density γ t(KN/m3)	17.5	17.4
Saturated unit density γ sat(KN/m3)	17.9	17.8
initial void ratio e0	—	0.9413
compression index Cc	—	0.29
Permeability K(cm/s)	1.0E-06	6.52E-06
Internal friction angle ϕ (°)	0	0
Cohesion C(KN/m2)	15.0	55.89

3. Pilot Project Design

3.1 Embankment Design

3.1.1 Basic Specifications

(1) Design Embankment Elevation

Generally, design elevation of embankment crest is determined based on the design high water level and the required freeboard for the embankment at the site considering the design flood frequency.

In Japan, the river planning is developed on condition that embankments are constructed at both sides in general. Therefore the design high water level of the river is determined based on the estimated water level at the design discharge predicted from the design rainfall at the design flood frequency. The design elevation of embankment crest is determined by adding a certain freeboard to design high water level mentioned above.

In case of the rivers in Bangladesh, it is difficult to determine the design high water level based on the results of runoff analysis, from the standpoints of following two aspects:

a. Upstream basins of most rivers in Bangladesh are located in India. Therefore, it is difficult to collect hydraulic and hydrological data in the upstream basins. In addition, it is also difficult to prepare the flood mitigation plan taken into consideration the development condition in the upstream basin.
b. Lengths of the rivers in Bangladesh are enormous, and many embankment failures occur due to violent bank erosion. As it is difficult to protect houses and properties from flood by constructing embankment along both sides of the rivers, villages and farm lands are protected by ring dyke in many cases.

Two (2) methods to determine the design high water level are described in the DSM separately in Bangladesh, The method of case A is used in case that the embankments are constructed on both side of river and flood water is confined within the river. The method of case B is used in case that the embankment is constructed on one side of river. The two cases are as follows:

Case A; Design high water level is estimated to be able to accommodate design discharge.
Case B; Design high water level are computed by frequency analysis of available annual maximum water level data collected at a water level gauging station near a planning site.

The planning of the Manu River corresponds to the case B as practiced for most rivers in Bangladesh. The design high water level is estimated based on probable water level estimated from the previous water level data of the neighboring water level gauging station.

(2) Basic Consideration for the Manu River

Embankment is constructed as continuous facilities along the stream direction of the river in order to protect the lives and property. Therefore crest elevation of embankments must be connected smoothly to the elevations of upstream and downstream embankments. In the case that embankments are constructed on both sides of river, artificial overflow could occur if the crest elevations of embankments on both sides are different as well.

Embankment reconstructed in the Pilot Project (“the Pilot embankment”) will be embedded in the short extent of existing embankment. The existing embankment of Manu River has been constructed since 1975. The design crest elevation of the embankment had been determined in each construction works in accordance with the DSM. However, it is difficult to clarify the design high water level and the design elevation of embankment crest of the Manu River, because there are not enough reports and drawings for the previous construction works.

In consideration of the above, it is considered reasonable and proper for determination of the crest elevation of the Pilot embankment as follows:

a. The crest elevation of the Pilot embankment is determined in consideration of the crest elevations of both of upstream and downstream embankments and that of embankment on opposite side.
b. Proposed design crest elevation is verified and evaluated with the probable high water level and appropriate free board in the Manu River.

Based on the survey results of the Project, the longitudinal profile of the Manu River around the Pilot Project Site is shown in Table-6.

The crest elevations of existing embankments on both upstream and downstream sides are around 13.00 m PWD. The design crest elevation of the Pilot embankment is proposed to set at 13.00 m PWD.

Table 6 Longitudinal Profile of River at the Pilot Project Site

						m PWD
Survey section No.	Distance(m)	Deepest River Bed Level	Crest Level Right Bank	Crest Level Left Bank	Ground Level Country side	Design crest Level Pilot Project
2	50	2.610	11.102	11.230	8.550	
3	100	3.570	12.007	12.484	10.010	
4	150	3.850	13.774	11.703	11.750	
5	200	3.190	12.809	12.888	9.460	
6	250	4.700	12.456	12.639	11.900	
7	300	4.260	12.788	12.733	10.118	
8	350	2.820	12.985	12.200	11.028	13.000
9	400	2.500	12.319	12.750	9.585	13.000
10	450	2.990	11.530	12.750	9.490	13.000
11	454	2.881	11.395	12.750	9.471	13.000
12	500	2.590	12.120	12.670	10.040	13.000
13	521	2.348	11.656	11.500	9.727	13.000
14	535	2.420	11.944	12.000	9.069	13.000
15	550	3.510	12.240	12.560	9.960	13.000
16	586	1.679	12.224	12.000	8.984	13.000
17	600	3.450	12.360	11.750	9.890	13.000
18	650	3.610	12.825	11.930	9.365	12.830
19	700	3.370	12.475	11.427	9.870	
20	750	3.300	12.571	11.210	9.990	
21	800	3.560	12.934	11.890	10.300	
22	850	3.930	13.408	11.750	10.480	
23	900	3.240	13.315	11.940	10.676	
24	950	3.480	13.305	11.940	10.290	
25	1000	2.720	12.850	11.945	9.950	
26	1050	3.330	12.845	11.990	9.830	
27	1100	3.310	13.075	12.040	10.350	
28	1150	2.970	13.214	12.050	9.988	

※Colored part shows the reconstruction section of the Pilot Project (around 265 m).

※ Distances in the table indicate the distance from the furthest downstream point (quantity surveying).

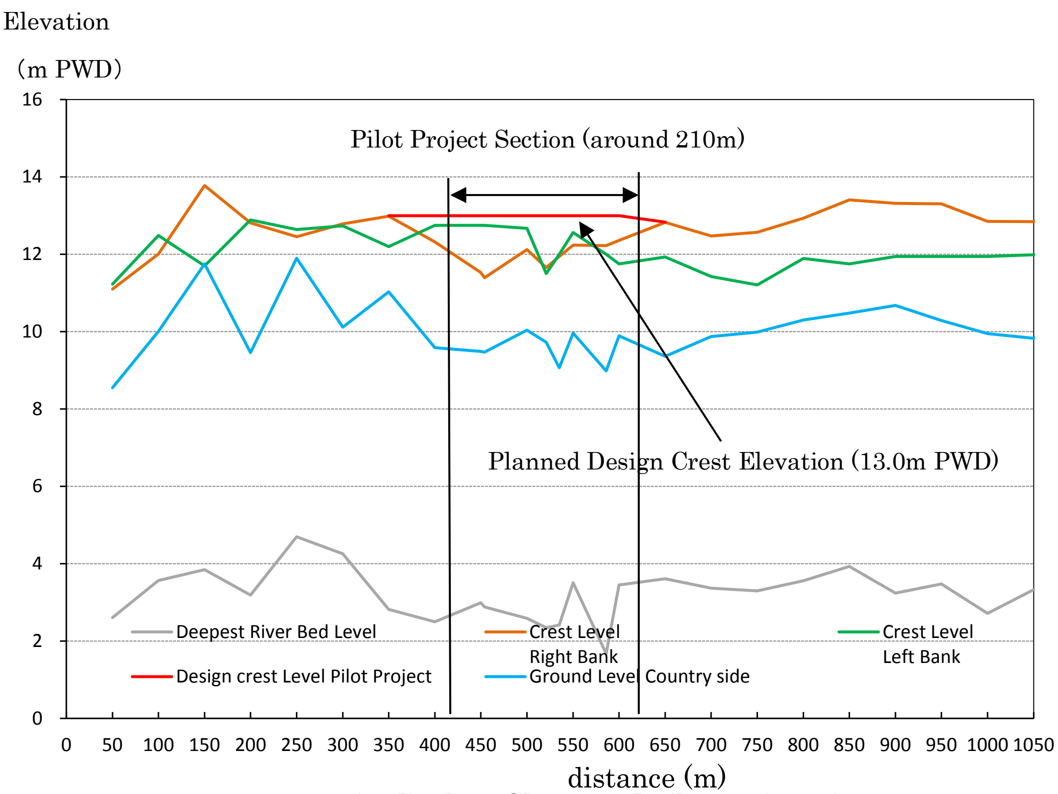


Figure 12 Longitudinal Profile around Pilot Project Site

(3) Evaluation of Design Elevation of Pilot Embankment

Due to lack of necessary data of the river cross sectional profiles, it is not able to estimate the equivalent water levels at the project site by using the non-uniform flow calculation. Therefore, the probable water levels at Moulvibazar Station is converted into the water level at the pilot project site, by use of the water surface gradient between the water level station and the pilot project site, described in the previous F/S Report as shown in below Figure:

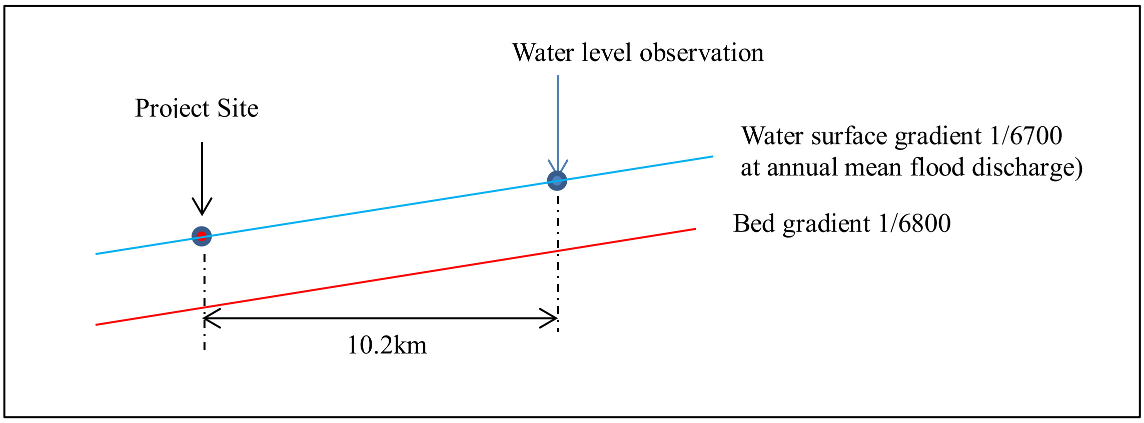


Figure 13 Longitudinal Profile between the Pilot Project Site and Moulvibazar Station

(Source: Final Report (Main Report), Feasibility Study for A Comprehensive Study for Drainage and Flood Management in the

Manu-Dhalai-Kushiyra-Khowai Systems, BWDB (Development Design Consultants Limited in association with Engineering & Planning Consultants Limited), June 2003

As presented in Figure-13, water surface gradient in the lower reaches of the Manu River is 1/6,666(≒ 1/6,700) at the annual mean flood discharge. The water level gap between the Moulvibazar Station and the Pilot Project site becomes 1.52m, that is calculated from the 10.2km distance and 1/6,700 water surface gradient. The probable water level at the pilot project site can be estimated by deducting 1.52 m from those at Moulvibazar Station.

Required crest elevation of the embankment at each probable water level is estimated by adding the freeboard for the embankment. Width of the channel at the pilot project site is less than about 300 m and there is no remarkable 'wind set-up' due to the meandering nature of the river. Therefore, the freeboard of the pilot of 0.9m is applied as stipulated in the SDM.

Estimation results are shown in Table-7 below:

Table 7 Probable Water Levels and Required Crest Elevation of Pilot Embankment

Return Period (year)	Possible water level at Station (m PWD) (A)	Probable Water Level at Pilot Project Site (m PWD) (B) = (A) – 1.52m	Required Embankment Crest Elevation (m PWD) (C) = (B) + 0.9m
2	12.35	10.83	11.73
3	12.64	11.12	12.02
5	12.87	11.35	12.25
10	13.06	11.54	12.49
20	13.17	11.65	12.55
30	13.21	11.69	12.59
50	13.25	11.73	12.63
80	13.27	11.75	12.65
100	13.28	11.76	12.66
150	13.30	11.78	12.68
200	13.31	11.79	12.69

Thus proposed crest elevation of the embankment is judged to be sufficient for expected flood.

(3) Evaluation of Embankment Elevation

According to these results, the proposed crest elevation (13.0 m PWD) of the pilot embankment is judged to be sufficient for expected flood.

(4) Design Low Water Level

The design low water level is used for calculating the mean water depth necessary for reviewing of embankment stability with respect to circular slip.

According to the Guidelines, design low water level is defined as follows. Since the target area is a non-tidal area, as a result of calculating the upper quartile of the Low Water Level (minimum daily water level between December~March over 10 years), this is 6.01 m PWD at Moulvibazar Water Level Gauging Station (SW202).

> Selection of DLWL:
>
> (I) Non Tidal Area
>
> - In order to make a balance between too high and too low value of LWL it is proposed that Upper Quartile value of annual LWL or a value in consideration of construction window be adapted as the DLWL for the design.
>
> (II) Tidal Area
>
> - Instead of annual lowest WL, annual lowest high tide (ALHT) should be used and then the Upper Quartile value of this may be used. Forecast of a fortnightly tide profile covering both neap tide and spring tide should also be provided so that the information may be used as an aid to under water construction planning.

Since the water level difference of the Pilot site and said water level gauging station is 1.52 m, the design low water level at the Pilot site is 6.01 m PWD - 1.52 m PWD = 4.49 m PWD.

Table 8 LWL between December ~ March (mPWD)

Year	Min LWL(mPWD)
1981	6.19
1982	6.08
1983	5.77
1984	5.51
1985	5.60
1986	5.64
1987	5.50
1988	5.45
1989	5.70
1990	5.64
1991	5.74
1992	5.88
1993	5.82
1994	5.65
1995	5.59
1996	5.58
1997	5.56
1998	5.66
1999	5.64
2000	5.83
2001	5.80
2002	6.21
2003	6.17
2004	6.22
2005	6.17
2006	6.04
2007	6.50
2008	5.95
2009	5.89
2010	5.76
2011	5.86
2012	5.95
2013	6.01
2014	5.99

Upper quartile
6.01mPWD

(5) Water Level during the Construction Period

According to the Cofferdam Installation Standard (Draft) (June 2010) of the Ministry of Land, Infrastructure and Transport, the following is stipulated regarding water level during the works period.

"The maximum water level during the works period over the past five years shall be adopted. However, if the said water level is deemed to be an abnormal flood over the past five years, the second highest water level over the past 10 years may be adopted."

Table 9 shows the top water level at the pilot site during the river works period (November ~ March) over the past five years (2009~2014) and the second highest water level over the past 10 years (2004~2014).

In the pilot site works, it is planned to drop gunny bags and foot protection blocks into the river from a barge in November and December, and to install the revetment foundation concrete on the mound built from gunny bags in January and February.

Therefore, in order to place the foundation concrete in January and February, the level of the bottom edge of the foundation concrete will need to be above the river water level.

Figure 5 shows the year-round trend of water level at the pilot site. The water level drops from November, however, in 2012, the drop in water level was delayed and the level was still high in January.

Accordingly, looking at the water level in January and February according to Table 9, omitting the highest water level for January 2012, the top water level over the past five years is 5.35, while the top two levels over the past 10 years are 5.31 and 5.35. Therefore, the standard water level during construction has been set at 5.35 m PWD ≒ 5.4 m PWD.

Table 9 Maximum Water Level at the Pilot Site over the Past 5 Years (2009~2014) and 10 Years (2004~2014)

	Top level (mPWD) in past 5 years	2nd level (mPWD) in past 10 years
November	6.79/ 2009	7.13/ 2008
December	6.45/ 2011	5.82/ 2010
January	6.16/ 2012	5.31/ 2014
February	5.35/ 2012	5.35/ 2012
March	6.22/ 2010	6.88/ 2007

(6) Representative Flow Velocity

The representative flow velocity Vo that is used in design of embankment and embankment protective works is sought by the following formula. The mean flow velocity Vm, which is obtained from Manning's formula of average flow velocity, is multiplied by a correction coefficient α that takes the influence of riverbed scouring on bends into account.

Representative flow velocity Vo

Vo=α・Vm (m/s)

Vm: Mean flow velocity (m/s)

α: Correction coefficient

(Coefficient that takes the influence of riverbed scouring on bends into account)

Formula for correction coefficient α: $\alpha = 1 + \frac{\Delta Z}{2Hd} + \frac{B}{2r}$ (Source: Design Manual, p32)

⊿Z: Scouring depth (m)

Hd: Design depth of water (m)

B: Width of low water channel (m)

r: Radius of curvature of bend (m)

1) Calculation of mean flow velocity Vm

Since there is no flow observation station close to the site and part of the catchment basin of Manu River is located in India, it is difficult to calculate the flow rate, flood water level and flow velocity in runoff analysis, etc. Therefore, the following Manning's formula of average flow velocity was used for calculating the representative flow velocity Vo needed for design.

■ Manning's formula of average flow velocity

$$V=\frac{1}{n}\times R^{2/3}\times I^{1/2}$$

V: Flow velocity (m/s)

n: Coefficient of roughness　0.028

R: Hydraulic radius (m) = Cross-sectional area of river/Wetted perimeter

I: Riverbed gradient 1/6800

Source: F/S for a comprehensive study for drainage and flood management in the Manu-Dhalai-Kushiyra-Khowai systems by BWDB, June 2003

Table 10 Results of Uniform Flow Calculation

Cross section No.	No.7	No. 8	No.9	No.10	No.12	No.15	No.17
n:Coefficient of roughness	0.028	0.028	0.028	0.028	0.028	0.028	0.028
S:Cross-sectional area of river (m2)	642.79	690.19	656.28	716.38	711.5	686.51	740.48
P:Wetted perimeter (m)	141.73	139.77	138.97	144.08	147.58	153.04	159.07
I:Riverbed gradient	0.000147	0.000147	0.000147	0.000147	0.000147	0.000147	0.000147
R: Hydraulic radius	4.5	4.9	4.7	5.0	4.8	4.5	4.7
Vm: Mean flow velocity (m/s)	1.19	1.26	1.22	1.26	1.24	1.18	1.21
Q:Discharge(m3/s)	762.6	866.6	799.9	903.7	879.2	808.6	893.9

※Cross-sectional area of river and wetted perimeter were obtained using CAD measurement, while the 11.65m described later was used for the HWL.

※Coefficient of roughness was set as 0.028 (case of Small channel on plain, with no grass according to Table 7 in the River and Erosion Control Standard).

※Concerning measurement point Nos.11.13.14.16, because the left bank isn't measured in the survey of damage conditions due to the purpose of examining sliding on the right bank, it isn't possible to measure the cross-sectional area of river or conduct other uniform flow calculation.

Based on the results of Table 10, the highest flow velocity of Vm=1.26m/s over the design section shall be the mean flow velocity Vm.

(Cross section No. 8, 10)

Table 11 Revised Ministry of Construction River and Erosion Control Standard (draft) 1997, Survey Section p132

	River or channel conditions	Scope of Manning's n
Artificial channel, improved river	Concrete artificial channel	0.014～0.020
	Spiral half-pipe channel	0.021～0.030
	Channel with stone masonry on both banks (mud bed)	0.025 (mean value)
	Bedrock excavation	0.035～0.05
	Bedrock forming	0.025～0.04
	Clay riverbed with flow velocity not enough to cause scouring	0.016～0.022
	Sandy loam, clayey soil loam	0.020 (mean value)
	Drag line dredging, little weeds	0.025～0.033
Natural river	Small channel on plain, with no grass	0.025～0.033
	Small channel on plain, with grass and shrubs	0.030～0.040
	Small channel on plain, with lots of grass and gravel bed	0.040～0.055
	Mountain channel, with gravel and boulders	0.030～0.050
	Mountain channel, with boulders and large boulders	0.040 or higher
	Large channel, with sandy bed and little meandering	0.018～0.035
	Large channel, with gravel bed	0.025～0.040

2) Setting of the coefficient of correction α

Since the site is situated on the outside of a bend in Manu River, the coefficient of correction α of the river bend is examined.

The coefficient of correction α is set according to the Design Manual, p32.

B: Low water channel 90 m (plan drawing measurement)

r: Radius of curvature 550 m (plan drawing measurement)

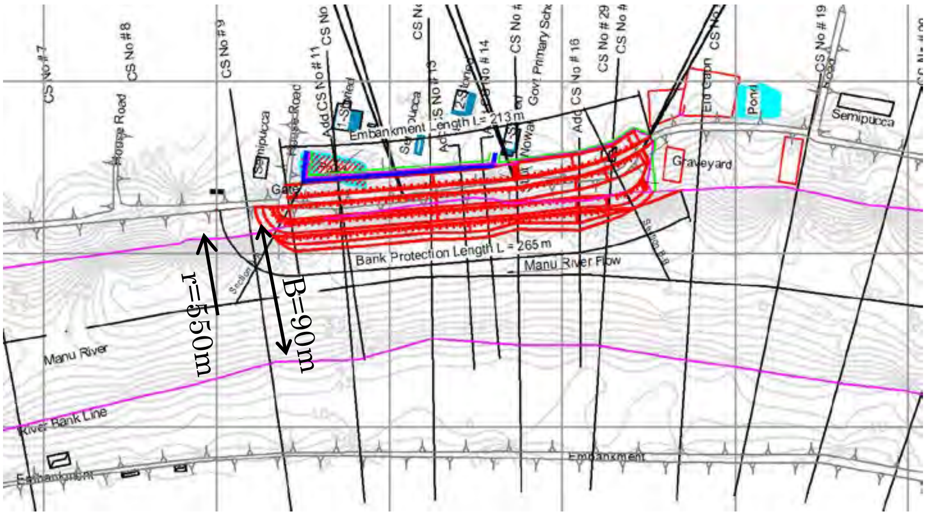


Figure 14 Pilot Site Low Water Channel and Radius of Curvature

Formula for correction coefficient α: $\alpha = 1 + \frac{\Delta Z}{2Hd} + \frac{B}{2r}$ (Source: Design Manual, p32)

⊿Z: Scouring depth (m)

Hd: Design depth of water (m)

B: Width of low water channel (m)

r: Radius of curvature of bend (m)

① Setting of the Scouring depth⊿Z(Ds)

In the Guidelines, the formula for calculating the scouring depth is given as follows.

$R = 0.47(Q/f)^{1/3}$ ・・・（Source: Guidelines p134）

$Ds = XR - h$

Ds : (m) Scour depth design discharge

Q : (m3/s) Design discharge

h : (m) Depth of flow, may be calculated as (HFL-LWL)

f : (-) Lacey's silt factor=1.76(d50)1/2

d50 : (mm) Median diameter of sediment particle

X : (-) Multiplying factor for design scour depth（below table)

Table 12 Multiplying Factors for Maximum Scour Depth by Lace'y approach

Nature of location	Factor(X)
Straight reach of channel	1.25
Moderate bend	1.50
Severe bend	1.75
Right angle abrupt turn	2.00
Noses of piers	2.00
Alongside cliffs and walls	2.25
Noses of guide banks	2.75

Below figure shows the level of deepest river bed of design range. Cross section No.16 is the deepest point and extremely low level, because after local scoring occurs, slips failure occurs due to local scoring. Therefore, Cross section No.16 exclude from scoring examination.

Table 12 shows multiplying factors each nature of location. This site is located moderate bend in the right bank. Therefore, multiplying factors has been set at 1.50.

Result of calculation shows next page. Level of scoring depth is 4.06mPWD.

On the other hand present deepest river bed is about 2.5mPWD, In the future, Riverbed will fall to 2.5m.Therefor; level of scoring has set 1.63m (Refer to Figure16)

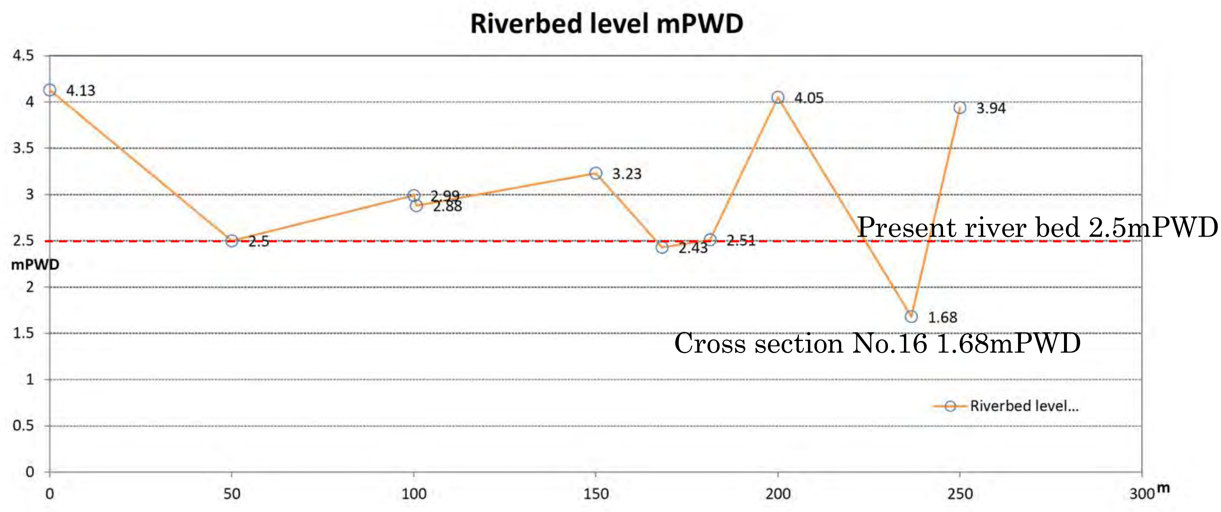


Figure 15 Elevation of deepest river bed（right side）

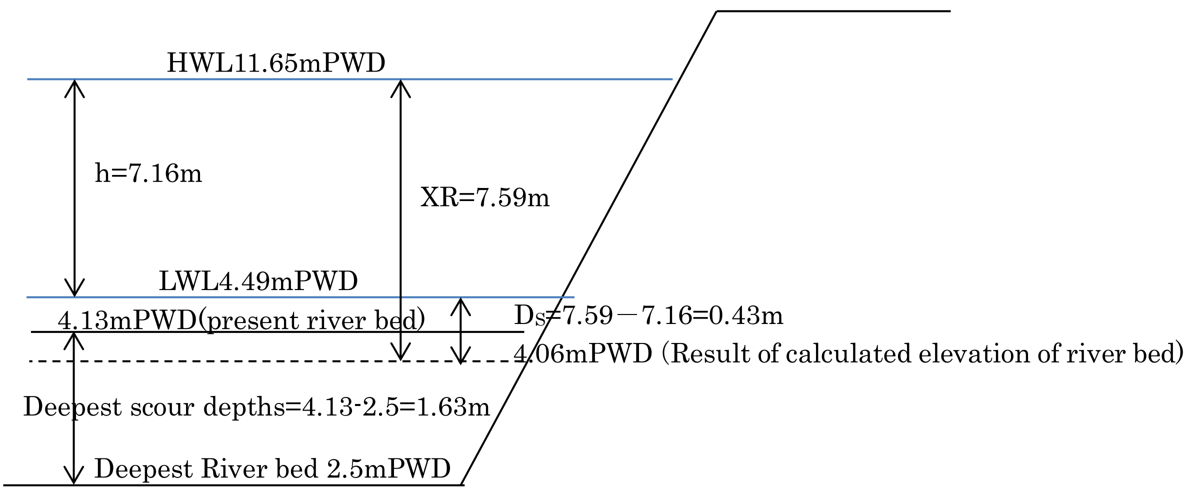


figure16 Result of calculation for scoring depth（standard place）

②Setting of design water depth Hd

Hd: Design water depth 9.15m (H.W.L.11.65mPWD-mean riverbed elevation 2.50 mPWD)

② Calculation of coefficient of correction α

$\alpha=1+1.63/(2*8.42)+90/(2*550)$

$=1.18<1.6$ (Dynamically Design Method on Revetment: upper limit value of α 1.6 on valley bottom plain, natural embankment, delta terrain)

3) Calculation of representative flow velocity

Vo: Representative Flow Velocity = $\alpha\times Vm=1.18\times 1.26 = 1.49\fallingdotseq 1.50$m/s

(7) Crest Width

Crest width is set at 4.30 m according to the Design Manual.

(8) Side Slope

Since stability is secured with 20% gradient according to the Design Manual, the side slope gradient is set at 20%.

(9) Elevation of Major bed of river

Since the current embankment has major bed and the direct level of embankment exceeds 10 m in parts, it is desirable to adopt compound cross section judging from the overall stability of embankment. In setting the elevation of major bed, the current elevation was organized as shown in Figure 17. Without reducing the current elevation of set-back too much, and upon taking the level of connection to the major bed at the edge into account, the design elevation of major bed was set at 9.1 m (PWD).

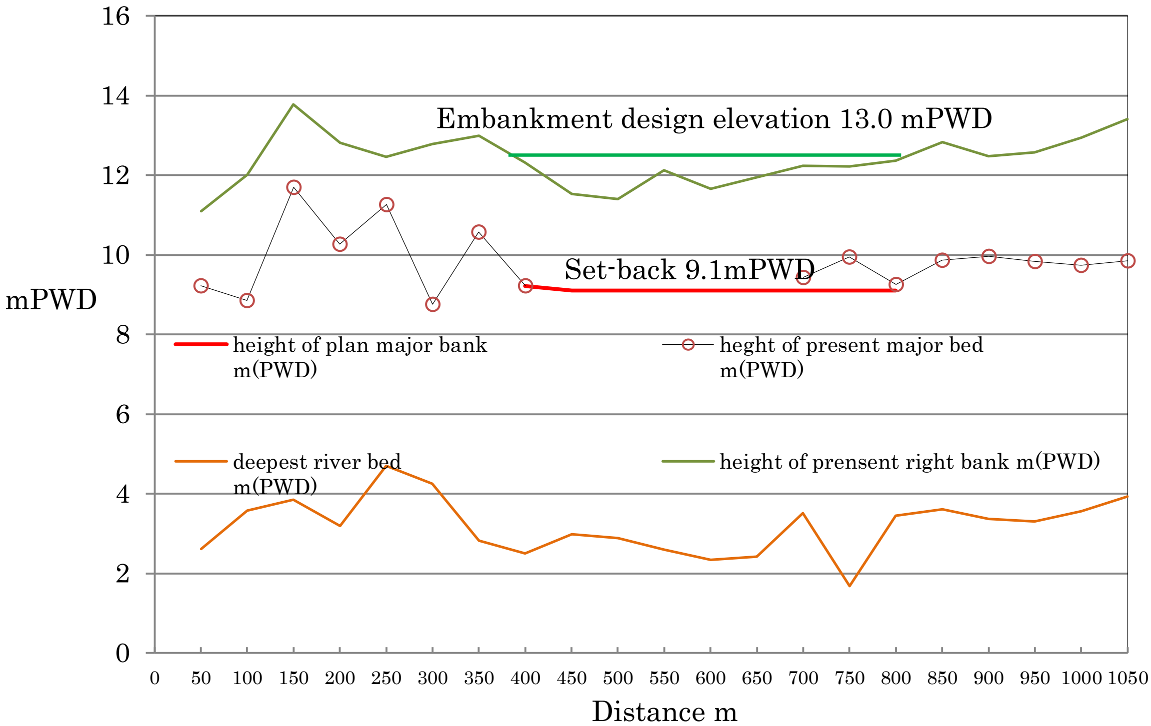


Figure 17 Pilot Site Longitudinal Profile

(10) Alignment of Embankment

As the pilot embankment is constructed as a part of existing embankment in a short time, alignment of the pilot embankment is designed in a manner fitted to that of existing embankment basically. In addition, the alignment of the pilot embankment is designed based on the following considerations:

a. There are graves on the set back at the upstream bend of the embankment. The extent of pilot embankment is designed excluding these graves.
b. There is an elementary school beside the pilot embankment. The alignment of pilot embankment must be designed without relocation of this school.
c. There is a house beside the pilot embankment. However this will move voluntarily and does not affect the alignment of pilot embankment.

The draft alignment of pilot embankment is shown in Figure-18.

Figure 18 Pilot Project Layout

3.1.2 Verification of Embankment Body Stability

Stability of the embankment is verified from the viewpoint of the following three items:

- Seepage analysis
- Circular slip
- Consolidation settlement

(1) Design Representative Cross Section

The design representative cross section is examined at measurement point No.16 where the difference between the crest elevation and front riverbed elevation is most highest (where stability is most at risk), in the design section. The following table shows the river side level difference and country side level difference in each cross section.

Table 13 Relationship between Crest Level and Frontal Riverbed Elevation

Measurement point No.	River side level difference (m)	Country side level difference (m)
7	5.53	2.67
8	8.74	1.93
9	10.42	3.41
10	10.01	3.9
11	10.12	3.9
12	9.85	3.23
13	10.57	3.04
14	10.43	3.9
15	8.97	3.31
16	10.92	4.12
17	9.17	3.54

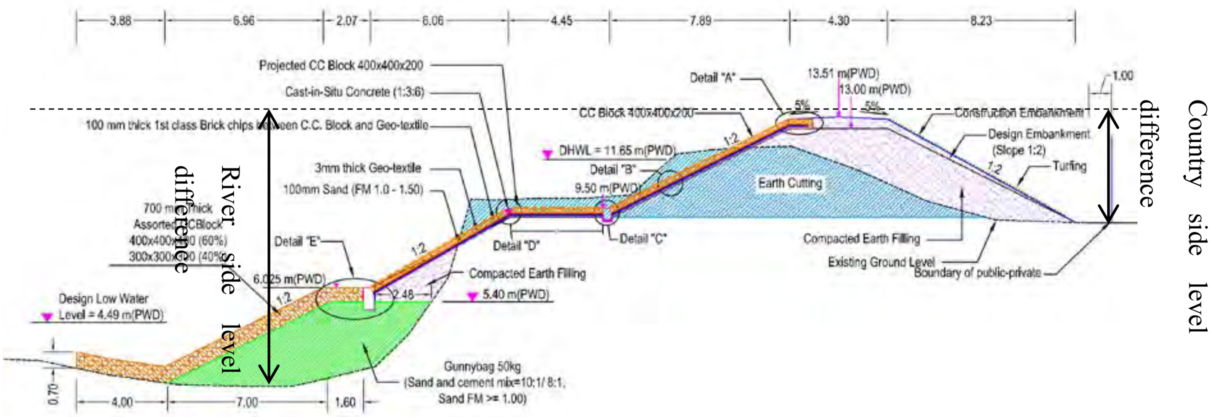
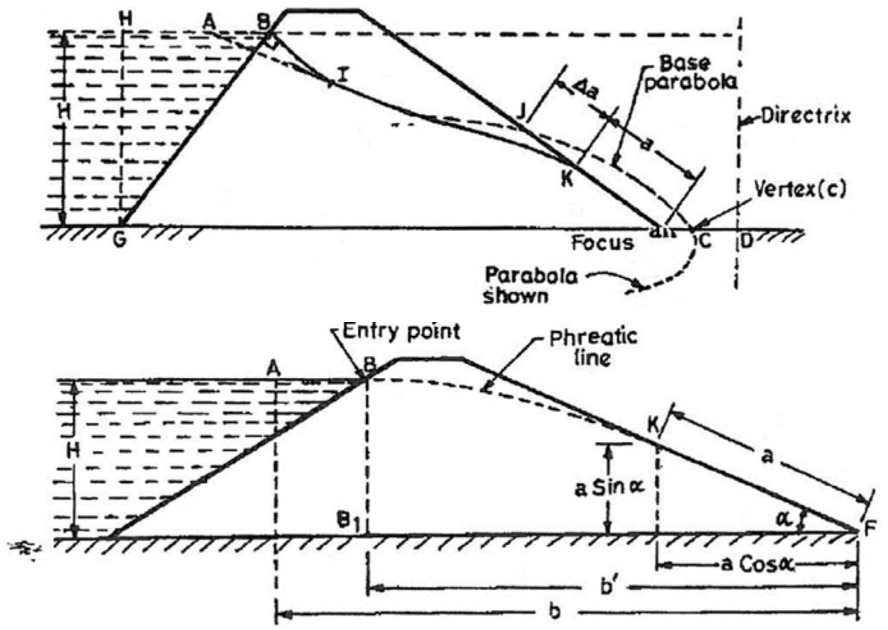


Figure 19 Cross Section of Measurement Point No.16

(2) Seepage Analysis

The Casagrande method is used to examine the seepage line. The following sections describe the concept and results of seepage line examination by the Casagrande method.

As a result, the seepage line appears in the embankment slope, however, since it is planned to use clay with very low permeability as the embankment material, there should be no seepage failure. Therefore, no seepage countermeasures will be implemented.



$\Delta a = (a+\Delta a)\ \frac{180° - \alpha}{400°}$

When a is 30°:

$$a = \frac{b'}{\cos\alpha} - \sqrt{\left(\frac{b'}{\cos\alpha}\right)^2 - \left(\frac{H}{\sin\alpha}\right)^2}$$

When a is30°~ 60°:

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2 cot^2\alpha}$$

Q = $\mathbf{k}$ ・S (**k**: coefficient of permeability)

S = $\sqrt{b^2 + H^2} - b$

Figure 20 Concept Diagram of Casagrande

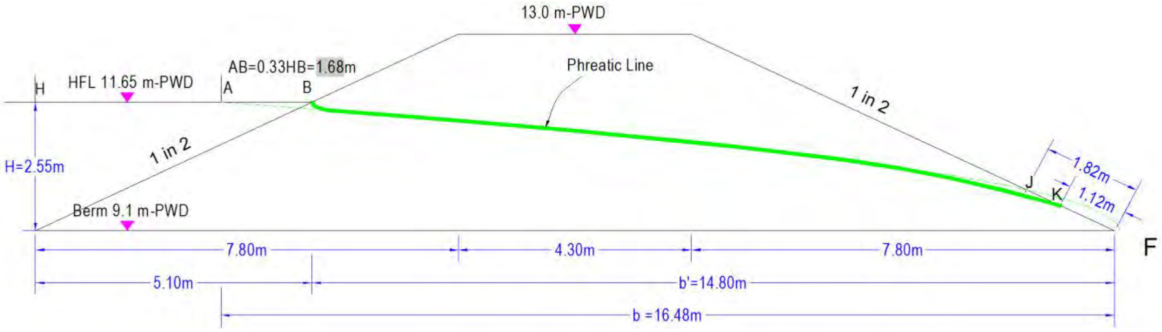


Figure 21 Seepage Flow Line (Design Cross Section No. 16)

Where S is the distance of the point(x,y) from the directrix, called focal distance.

The equation of base parabola is given by $\sqrt{x^2 + y^2} = x + S$

AB ≈ 0.33 HB =0.33 * 5.10 = 1.68

x =	16.48
y =	2.55

$$S = \sqrt{x^2 + y^2} - x$$

S = **0.196**

A few more coordinate of base parabola at known distance (x) are orked out in the following table using,

$$y = \sqrt{S^2 + 2xS}$$

x	y² = 2xS+S² = 0.392x + .154	y
0.00	0.038	0.196
-0.07	0.011	0.105
0.00	0.038	0.196
0.50	0.235	0.484
1.00	0.431	0.656
2.00	0.823	0.907
3.00	1.215	1.102
4.00	1.607	1.268
5.00	2.000	1.414
6.00	2.392	1.547
7.00	2.784	1.669
8.00	3.176	1.782
9.00	3.569	1.889
10.00	3.961	1.990
11.00	4.353	2.086
12.00	4.745	2.178
13.00	5.138	2.267
14.00	5.530	2.352
15.00	5.922	2.434
16.00	6.314	2.513
16.48	6.503	2.550

At entry, the phreatic line is started from the point B in such a way that it becomes at right angles to the u/s face of the embankment. A reverse curvature is, given as shown in above figure. At exit, the point K at which the phreatic line intersects the d/s face can be easily obtained by using

$$\Delta a = (a + \Delta a)\left[\frac{180 - \alpha}{400}\right]$$

Where, $\tan \alpha = \frac{1}{2}, or \ \alpha = 26^{0}.54$

$(a + \Delta a) =$ Distance FJ, i.e; the distance of the focus form the point at which the base parabola intersects the d/s face

= **1.82**

$\alpha =$ 26.57 Slope 1: **2.0**

$\Delta a =$ 1.82*((180-26.57)/400) = 0.70

a= 1.82-0.70 = **1.12**

(3) Examination of Circular Slip

The Bishop method using the set soil constants is used to examine circular slip. As indicated in the following table, the results of examination show that the safety factor is secured and there is no issue with safety in the design cross section.

$$F_s = \frac{R\Sigma(\{c.l\cos\alpha + (1+Kv)W'\tan\varphi\}/m\alpha)}{\Sigma\ \{(1+Kv)W\cdot x + Kh\cdot W\cdot y + P\cdot a\}}$$

$$m_\alpha = \cos\alpha + \frac{\tan\varphi'\cdot\sin\alpha}{Fs}$$

Fs: Safety factor

C: Adhesive force of soil (kN/m2)

φ: Internal angle of friction of soil (°)

l: Length of slice circle (m)

W': Effective weight of slice (KN/m)

W: Weight of slice (kN/m)

Kh: Design horizontal seismic coefficient (not considered in this case)

Kv: Design vertical seismic coefficient (not considered in this case)

R: Radius of circle (m)

α: Angle of inclination of the slice base (°)

x: Horizontal distance between the slice center of gravity and center of circle (m)

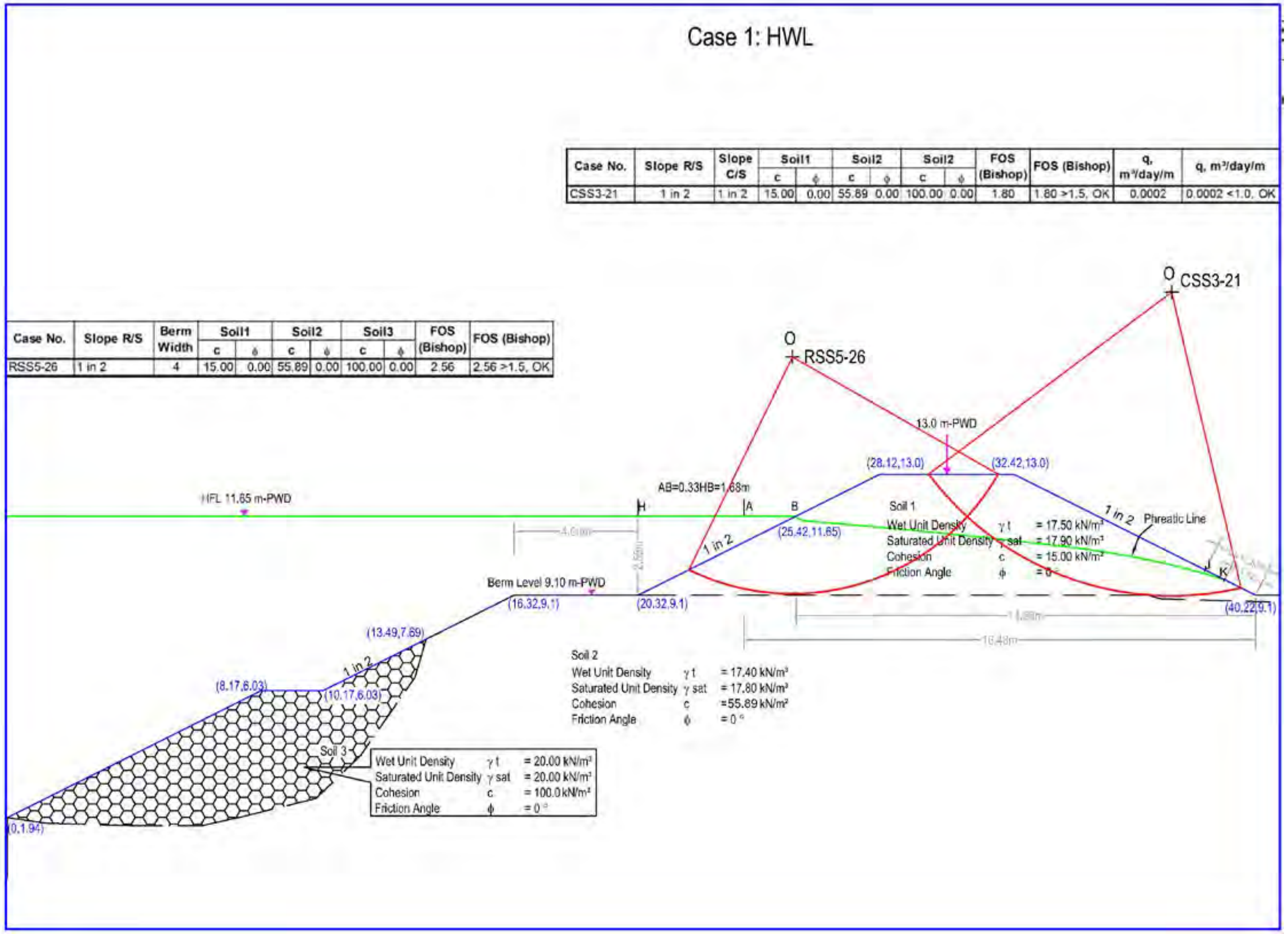
y: Vertical distance between the slice center of gravity and center of slip circle (m)

P: Water pressure acting on earth clump inside the slip circle (KN/m)

α: Vertical distance of horizontal line passing through the pressure point and center of the slip circle (m)

Table 14 Calculation Cases and Results

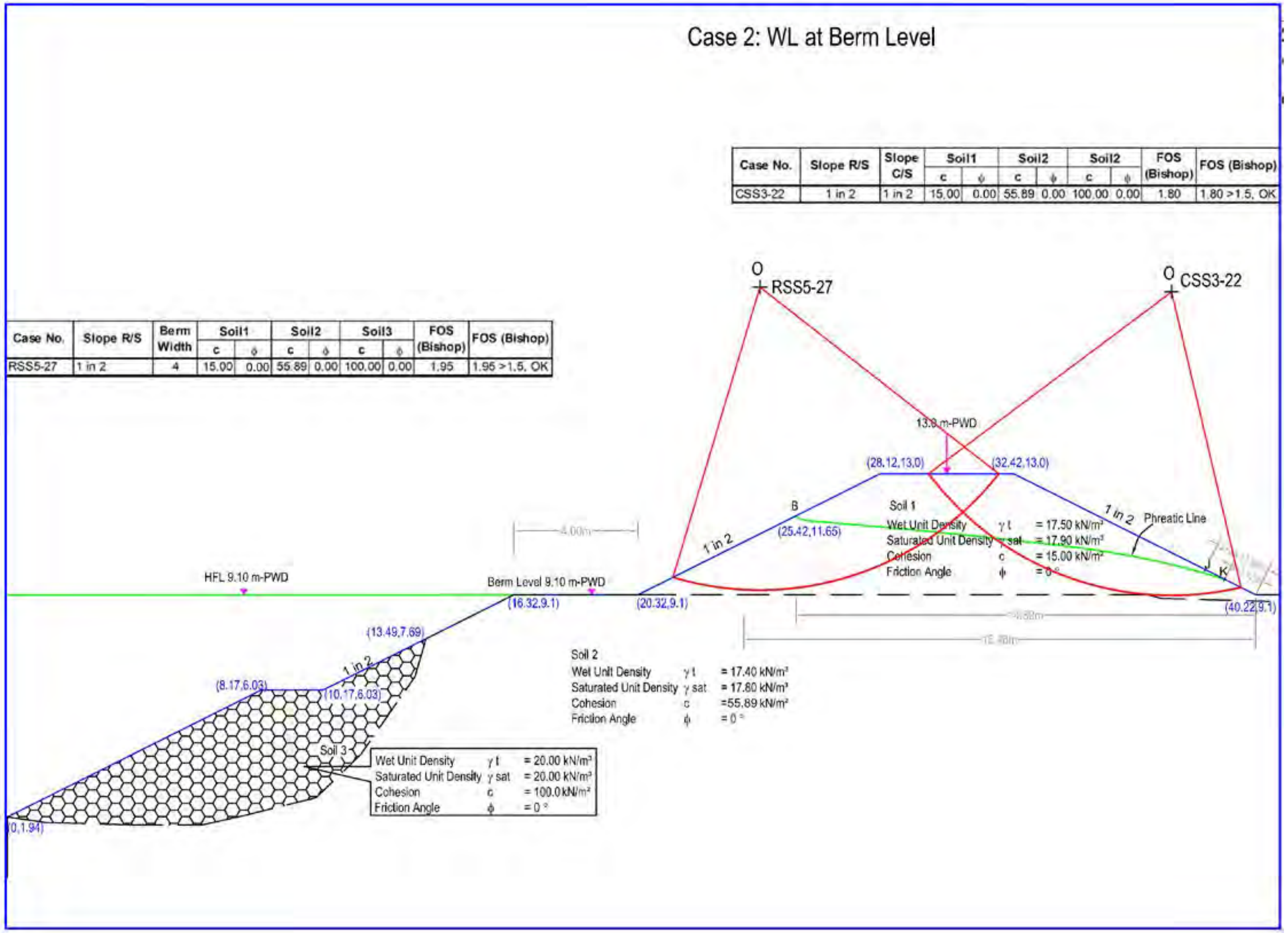
Calculation case	Frontal water level	Safety factor	
		River side	Country side
H.W.L.	11.65mPWD	2.90>1.50 OK	2.04>1.50 OK
After Flood	9.10mPWD	2.20>1.50 OK	—
L.W.L.	4.49mPWD	2.22>1.20 OK	—



Case No.	Slope R/S	Slope C/S	Soil1		Soil2		Soil2		FOS (Bishop)	FOS (Bishop)	q, m³/day/m	q, m³/day/m
			c	ϕ	c	ϕ	c	ϕ				
CSS3-21	1 in 2	1 in 2	15.00	0.00	55.89	0.00	100.00	0.00	1.80	1.80 >1.5, OK	0.0002	0.0002 <1.0, OK

Case No.	Slope R/S	Berm Width	Soil1		Soil2		Soil3		FOS (Bishop)	FOS (Bishop)
			c	ϕ	c	ϕ	c	ϕ		
RSS5-26	1 in 2	4	15.00	0.00	55.89	0.00	100.00	0.00	2.56	2.56 >1.5, OK

Figure 22 Calculation Case 1: Frontal water level at H.W.L. 11.65mPWD



Case No.	Slope R/S	Slope C/S	Soil1		Soil2		Soil2		FOS (Bishop)	FOS (Bishop)
			c	ϕ	c	ϕ	c	ϕ		
CSS3-22	1 in 2	1 in 2	15.00	0.00	55.89	0.00	100.00	0.00	1.80	1.80 >1.5, OK

Case No.	Slope R/S	Berm Width	Soil1		Soil2		Soil3		FOS (Bishop)	FOS (Bishop)
			c	ϕ	c	ϕ	c	ϕ		
RSS5-27	1 in 2	4	15.00	0.00	55.89	0.00	100.00	0.00	1.95	1.95 >1.5, OK

Figure 23 Calculation Case 2: Frontal water level at 9.1mPWD

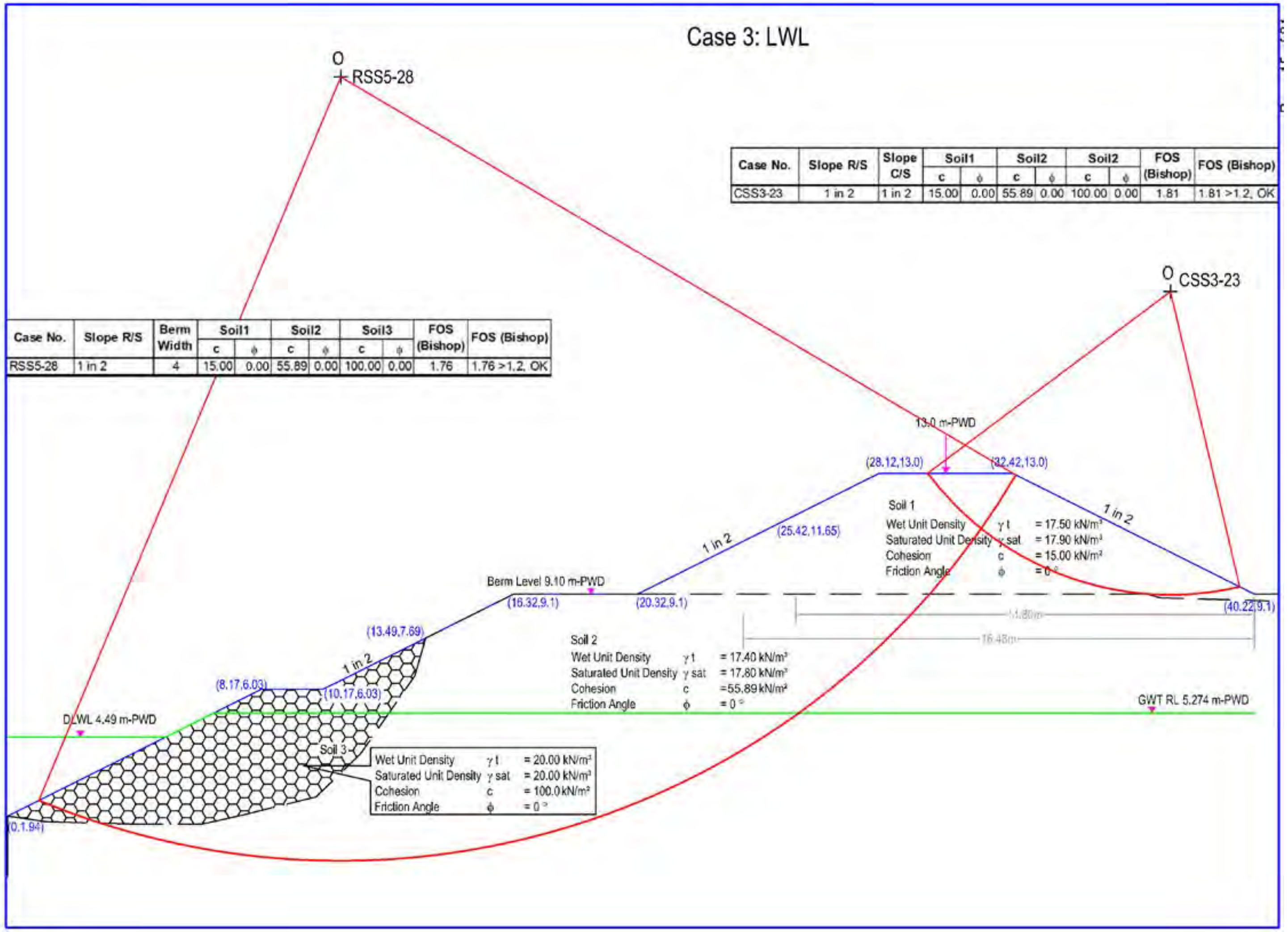


Figure 24 Calculation Case 3: Frontal water level at D.L.W.L. 4.49mPWD

(4) Examination of Consolidation Settlement

In the Design Manual, the Terzaghi formula is used to examine consolidation subsidence. In conducting examination, the set soil constants are used. As a result, since subsidence of 39 cm is confirmed, extra banking of 39cm ≒ 40.0cm will be added to compensate.

$$S = \frac{C_c \cdot H}{1+e_0} \log_{10} \frac{P_0+\Delta P}{P_0}$$

S: Amount of subsidence of single layer

Cc: Consolidation index

H: Compaction layer thickness

e_o: Initial void ratio

Po: Effective overburden stress before embankment construction

⊿P: Increase in overburden stress after embankment construction

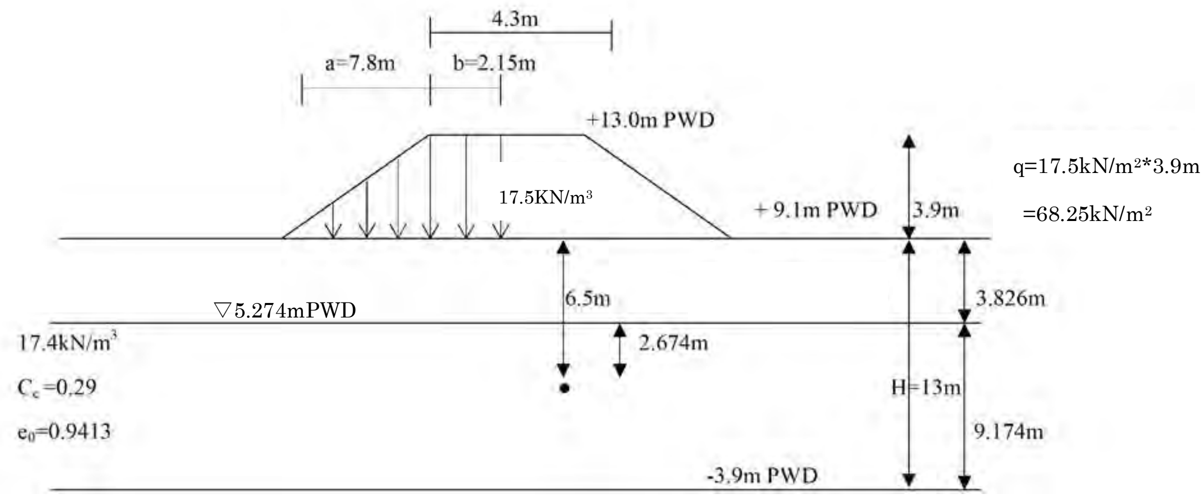


Figure 25 Model Diagram of Consolidation Calculation

$$P_0 = 17.4kN/m^3 * 3.826m + (17.4 - 9.81)kN/m^3 * 2.674$$

$$= 86.87kN/m^2$$

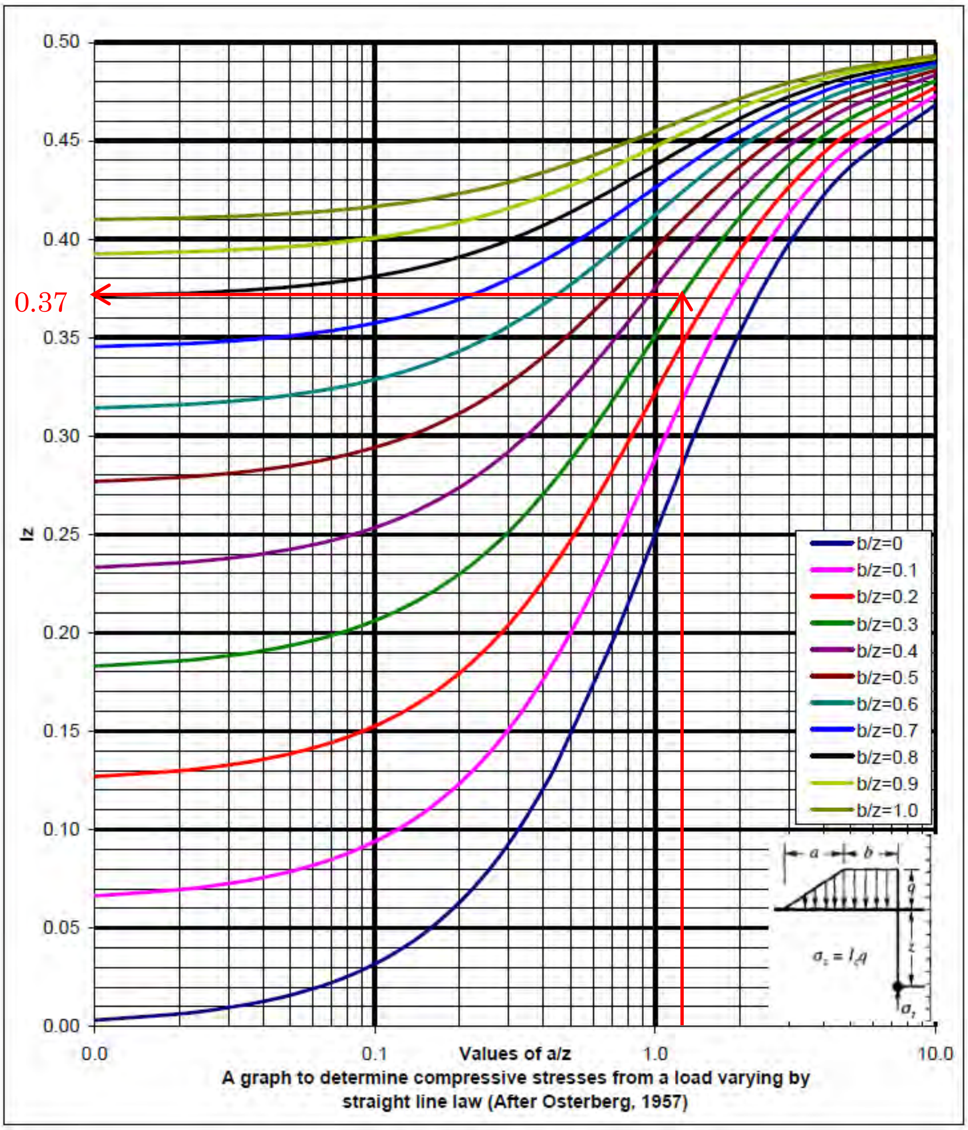


Figure 26 Relationship between Increased Stress and Depth (Osterberg,1957)

$$\frac{a}{z} = \frac{7.8m}{6.5m} = 1.2$$

$$\frac{b}{z} = \frac{2.15m}{6.5m} = 0.33$$

According to the Osterberg figure, Iz=0.37

Concerning Iz at depth of 6.5 m, since load is considered on the left and right sides of the embankment, Total Iz = 0.37*2=0.74.

$$\Delta P = I_z \times g = 0.74 \times 68.25 KN/m^2 = 50.51 KN/m^2$$

$$S = \frac{C_c H}{1+e_o}\log\left(\frac{P_0+\Delta P}{P_0}\right)$$

S: Amount of subsidence of single layer

Cc: Consolidation index = 0.29 (result of indoor testing)

H: Compaction layer thickness = 13m

e_o: Initial void ratio = 0.9413 (result of indoor testing)

Po: Effective overburden stress before embankment construction

＝86.87KN/m^2

⊿P: Increase in overburden stress after embankment construction

＝49.64KN/m^2

$$s = \frac{0.29*13}{1+0.9413}\log\left(\frac{86.87+50.51}{86.87}\right)$$

$$= 1.94*0.199$$

$$= 0.39m$$

Therefore, the extra banking shall be 0.39m = 0.4m.

3.2 Revetment for Embankment Slope

3.2.1 Revetment for Embankment Slope

In the pilot design, facing blocks are planned for embankment protection, and foot protection work is planned for scouring prevention. The respective comparison sheets are shown on the following pages.

(1) Slope Block Work

As a result of the comparative examination, it was decided to adopt projected CC blocks as the slope protection work. The comparison sheet is shown on the following page. On the outer bank of a bend in a meandering river, there is a tendency for flow velocity to accelerate in the longitudinal direction and local scouring to be caused by secondary flows in the transverse direction (eddies at right angle to the river flow). Therefore, projected CC blocks are planned in order to reduce the flow velocity and limit occurrence of secondary flows.

(2) Foot Protection Work

As a result of the comparative examination, it was decided to adopt the existing CC blocks used in BWDB works for foot protection. The comparison sheet is shown on the following page.

In Japan, various types of foot protection blocks are used in order to prevent scouring, however, because the flow velocity of Manu River is no higher than 2m/s and the budget is limited, the pre-existing local method of randomly placed CC blocks is planned.

(3) Front back Fill Materials

The front of the current embankment at this site is largely collapsed. In the Pilot Project, as a means of improving the work method, it is necessary to perform compaction on each layer. For this reason, the layer underneath the compaction work must comprise firm ground or materials.

In Japan, it is normal for dry work to be carried out with the aid of coffering, however, in Bangladesh, since coffering is expensive and difficult to implement in deep water, geo-bags or gunny bags filled with sediment are usually deposited from barges.

However, due to the poor durability of gunny bags, because there is concern that the bags will break and allow the sand to escape, as a result of discussion with the BWDB design department, it has been decided to fill bags with a cement-sand mixture.

In Bangladesh, since gunny bags (cement-sand) have a proven record of use not as the backfill material as planned in the Project but rather as slope protection work following disasters, it is thought there will be no problem in using them as frontal filling materials. Moreover, even if the gunny bags become deteriorated over time, the contents will not spill out and it will still be possible to implement sure compaction work.

Table 15 Slope Revetment Work Comparison Sheet

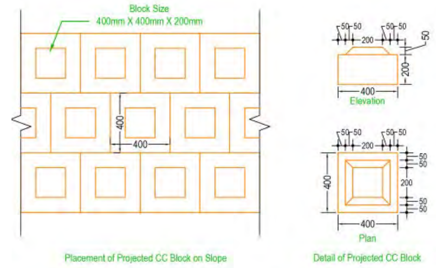
	Natural stones (concrete packing)	Natural stones (dry masonry)	Gabion matting	CC blocks	Projected CC blocks
Image view					
Features	Revetment is built by concrete-packed placement of natural stones extracted from the Silett district. Since concrete is packed around the stones, smaller diam stones than in the case of dry masonry can be used.	Revetment is built by dry masonry placement of natural stones extracted from the Silett district. Compared to the option on the left, stone diam need to be larger.	Revetment is built by packing natural stones into a wire cage.	This is the CC block method used by BWDB in conventional river works. Because the surface is smooth, resistance is weak during flooding.	Improved CC blocks with projections are used in the conventional BWDB approach. It is anticipated that such projected blocks can slow the flow velocity and limit the occurrence of secondary flows.
Economy	Not feasible because natural stones are not readily available.	Not feasible because natural stones are not readily available.	Not feasible because natural stones are not readily available.	11,160Tk/m^3 (400*400*200)	11,220Tk/m^3 (400*400*200 with projection)
Ease of execution	There is no problem over the scope in which materials can be transported by manual labor.	There is no problem over the scope in which materials can be transported by manual labor.	There is no problem over the scope in which materials can be transported by manual labor.	Based on past performance, there is no problem.	The projection is simple and blocks can be made by adding projected formwork.
Applicability	Large quantities of natural stones are not readily available.	Large quantities of natural stones are not readily available.	Large quantities of natural stones are not readily available.	Based on past performance, there is no problem.	Projected blocks are used in other countries too without any problems.
Evaluation	×	×	×	△	○

Table 16 Foot Protection Work Comparison Sheet

	Natural stones	Bag-filled foot protection	Fascine mattress	CC blocks	Geo-bags/Gunny bags
Image view					
Features	Revetment is built by concrete-packed placement of natural stones extracted from the Sylhet district. Through packing concrete between the stones, the diam of the stones can be smaller than in the case of dry masonry.	Bags filled with natural stones are used as the foot protection work. This method is adopted in Japan.	In this traditional Japanese method, a box frame is made with sticks, and this is filled with stones.	This is the CC block method used by BWDB in conventional river works. CC blocks of differing specifications are randomly piled.	Revetment is built using the bags that are used by BWDB in conventional river works.
Economy	Not feasible because natural stones are not readily available.	Not feasible because natural stones are not readily available.	Not feasible because natural stones are not readily available.	1,1000Tk/m^3 (300*300*300) 10,800Tk/m^3 (400*400*400)	106Tk/m^3(50kg)
Ease of execution	There is no problem over the scope in which materials can be transported by manual labor.	There is no problem over the scope in which materials can be transported by manual labor.	There is no problem over the scope in which materials can be transported by manual labor.	Based on past performance, there is no problem.	Based on past performance, there is no problem.
Applicability	Large quantities of natural stones are not readily available.	Large quantities of natural stones are not readily available.	Large quantities of natural stones are not readily available. The locally available eucalyptus timber is hardwood and not suitable for this work.	Based on past performance, there is no problem.	There is concern over deterioration over time caused by ultraviolet rays and running water abrasion. There is no problem if bags are used as backfill materials and are not affected by ultraviolet rays and running water abrasion.
Evaluation	×	×	×	○	△

3.2.2 Design of Revetment for Embankment Slope

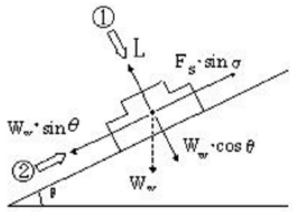
(1) Design of Slope Block Work

Since the Guideline, which is the standard for revetments in Bangladesh, does not deal with the concepts of drag coefficient and up lift coefficient, examination of the facing blocks for use in slope protection is conducted based on the “Sliding-Unit” model according to the Design Manual. The basic formula for verifying stability regarding fluid force is shown below.

$$\mu\,(W_w \cdot \cos\theta - L) \geqq ((W_w \cdot \sin\theta)^2 + D^2)^{1/2}$$

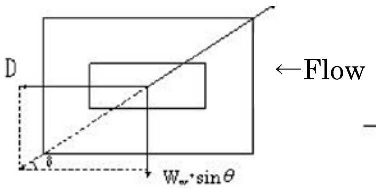
$$L = \rho_w/2 \cdot C_L \cdot A_b \cdot V_d^2 \text{ (kgf)【N】}$$

$$D = \rho_w/2 \cdot C_D \cdot A_D \cdot V_d^2 \text{ (kgf)【N】}$$

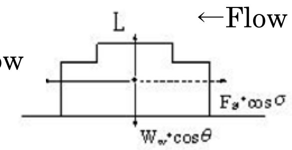


This shows the condition of a block placed on the right bank

Force viewed from the direction of ①



Force viewed from the direction of ②



Slope maximum in cline direction

μ : Coefficient of friction μ=0.65 (coefficient of friction between soil and suction prevention material)

W_w : Weight of blocks in water (kg)

θ : Slope gradient

ρ_b : Density of blocks 2250(kg/m^3)

ρ_w : Density of water ρ_w=980 (kg/m^3)

g : Acceleration of gravity (9.81m/s^2)

A_b : Projected area of blocks when viewed from above (m^2)

A_D : Projected area of blocks pertaining to drag (m^2)

t_b : Projected area of blocks pertaining to drag (m)

C_L : Uplift coefficient of blocks

C_D : Uplift coefficient of blocks

V_d : Design flow velocity at level t_b (m/s)

$$V_d = \frac{8.5+5.75log10\left(\frac{tb}{ks}\right)+2}{6.0+5.75\log10(\frac{Hd}{ks})} \cdot V_o$$

Hd: Design depth of water

ks: Equivalent roughness 0.08

Vo: Design flow velocity Vo＝αVm

Vm: Mean flow velocity (calculated from Manning's formula of average flow velocity)

Vm＝1/n・Hd2/3・Ie1/2

n: Manning's coefficient of roughness

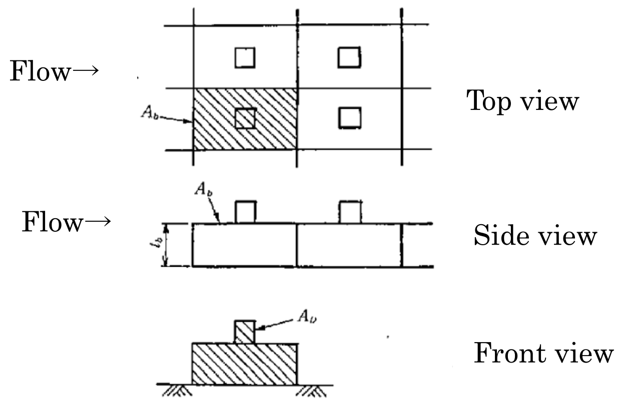
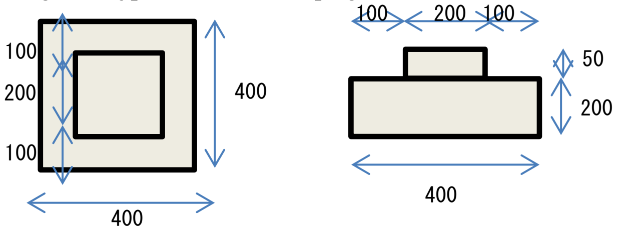


Figure 27 Source: Design Manual, pp31～33

1) Setting of drag coefficient and uplift coefficient

Since 400×400×200 CC blocks are used as embankment protection works on the upstream and downstream of Manu River, this shape with an added projection has been set for the Pilot Project. Stability verification is conducted and the specifications are determined assuming this shape.

Table 17 Specifications of CC Block Projection

	Unit
	Projected type400×400×200（projection 50） 
Drag coefficient C_D	0.7 (Refer to Design Manual, p32)
Lift coefficient C_L	0.15 (Refer to Design Manual,p32)
Ks: Equivalent roughness	0.08 (Refer to Design Manual,p32)
Ab: Projected area of blocks when viewed from above	0.4×0.4=0.16
A_D: Projected area pertaining to drag	0.4×0.2＋0.05×0.2=0.09
Weight	(0.4×0.4×0.2＋0.2×0.2×0.05) ×2.25=76.5kg

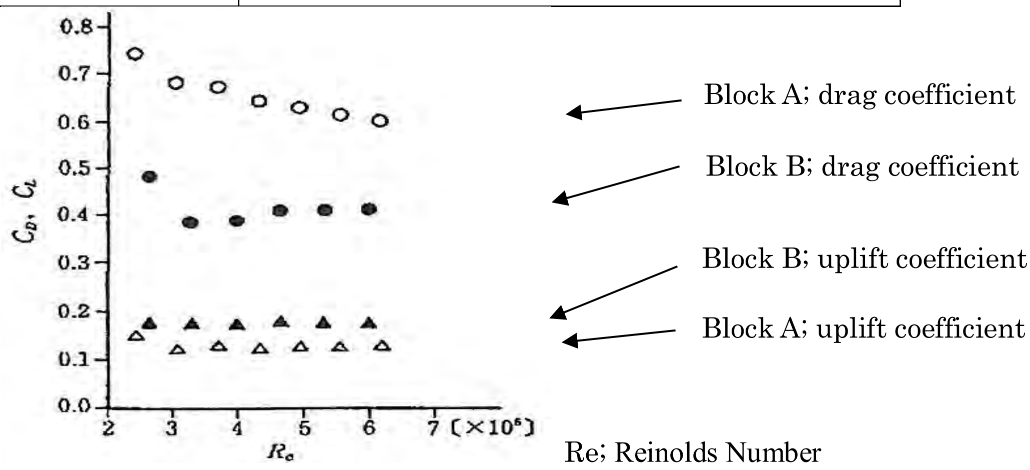


Figure 28　Coefficients by Type of Unit Block (Source: Design Manual, p34)

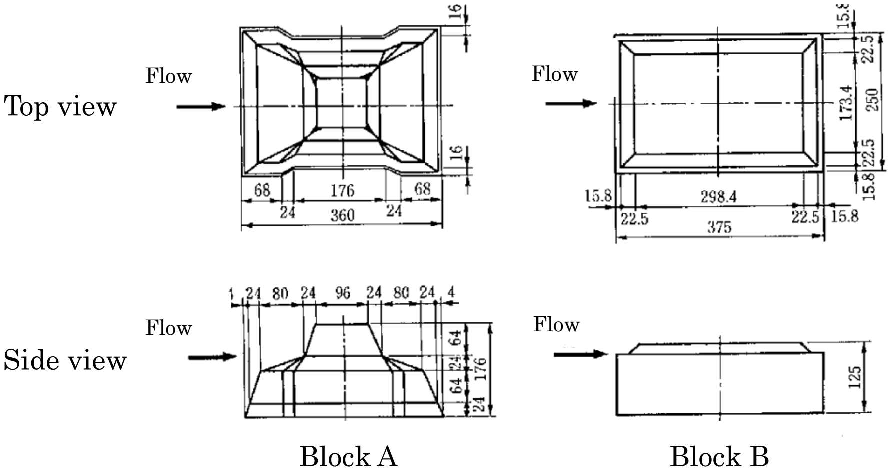


Figure 29 Block Standard Drawings (Source: Design Manual, p34)

2) Design conditions

Design flow velocity: 1.50m/s

H.W.L.: 11.65mPWD

Slope gradient: 1:2.0

Design cross section: Measurement point No.16

Deepest riverbed elevation: 1.68mPWD

Design water depth: H.W.L.11.65mPWD - Deepest riverbed elevation 1.68mPWD = 9.97m

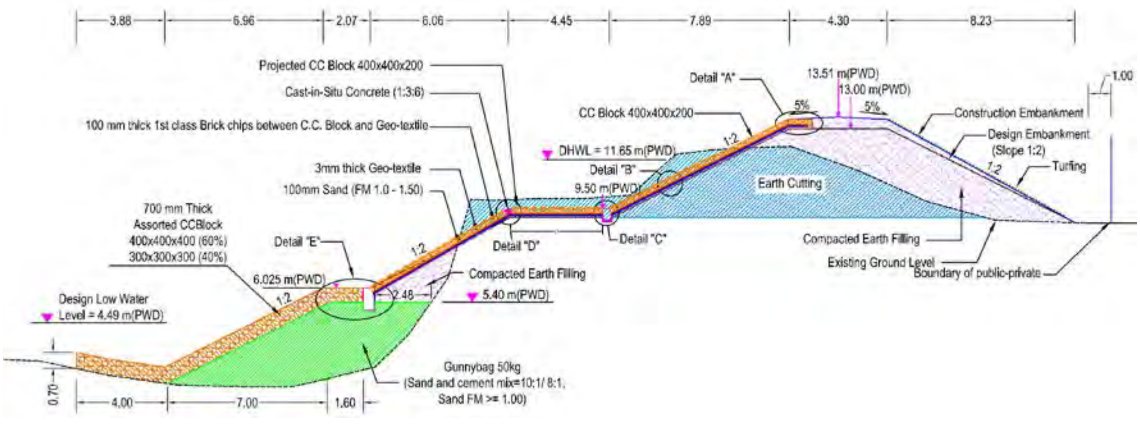


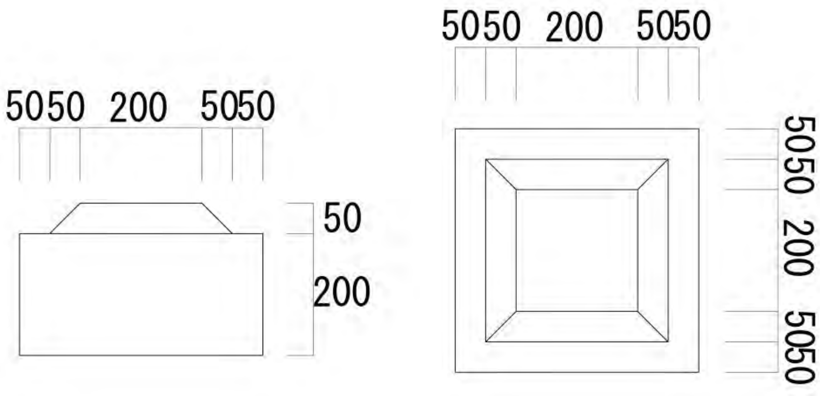
Figure 30 Design cross section: Measurement point No.16

3) Results of stability verification

As a result, the required weight is calculated as 23.76 kg. The calculation sheet is shown in the annex.

In other parts of Manu River, apart from the Project, BWDB has designed and placed blocks with dimensions of 400×400×200 (thickness), and since there are no reports of such blocks being damaged, blocks of similar size (400×400×200, with projections) that weigh 76.5kg and can be placed manually will be adopted.

4) Final specifications of Projected CC blocks



Side view Top view

Figure 31 CC Block with Projection in the Pilot Project

(2) Design of Foot Protection Work

In examining the foot protection blocks, the Bangladesh Guidelines for River Bank Protection (2010 BWDB), p142 will be followed.

$$D_n \geq \frac{0.035 * u^2}{\Delta_m * 2g} * \frac{\phi_{sc} K_\tau K_h}{K_s * \psi_{cr}}$$ (Source: Guideline p142)

Where,

D_n	(m)	Equivalent diameter (cover layer)
Δ_m	(-)	$(\rho_s - \rho_w)/\rho_w$ = relative density of submerged material
ρ_s	(kg/m^3)	Ddensity of protection material
ρ_w	(kg/m^3)	Density of water
$\bar{u}$	(m/s)	Depth averaged mean flow velocity
g	(m/s^2)	Acceleration due to gravity
ϕ_{sc}	(-)	Stability factor for current
K_τ	(-)	Turbulence factor
K_h	(-)	Depth factor, dependent on the assumed velocity profile and water

		depth (h) to equivalent roughness level ratio
K_s	(-)	Slope parameter
Ψ_{cr}	(-)	Critical shield's parameter
α	(°)	Slope angle of bank or structure
θ	(°)	Angle of repose considering the material specific internal friction

Design of Bank Protective Works
at Pilot Project Site along Right Bank of Manu River

Target Cross Secttion No. 16

Design data:

1. Highest flood level				=	11.65 m-PWD
2. Free board				=	0.90 m
3. Crest level (Top level)				=	12.55 m-PWD
				≈	13.00 m-PWD
4. Berm level				=	9.10 m-PWD
5. Design Low Water Level				=	4.49 m-PWD
6. Average river bed level				=	3.056 m-PWD
7. Revetment material : C. C. Block & Gabions filling by stone					
8. Bank slope above berm level, α	= 1V:2H	=	1: 2.0	α =	26.57 ^{0}
9. Bank slope below berm level, α	= 1V:2H	=	1: 2.0	α =	26.57 ^{0}
10. Scour slope below bed level, α	= 1V:2H	=	1: 2.0	α =	26.57 ^{0}

CC blocks, concrete with stone aggregate (dumping below construction water level)

D_n =	Cubic dimension of CC Blocks		
Δ_m =	Relative density of submerged material = $(\rho_s - \rho_w) / \rho_w$	=	1.30
ρ_s =	Density of CC block	=	**2250** kg/m³
ρ_w =	Density of water	=	980 kg/m³
u =	Flow velocity	=	**1.50** m/sec
g =	Acceleration due to gravity	=	9.81 m/s²
$\varnothing_{sc}$ =	Stability factor for current (Continuous protection)	=	**0.8**

	(Guideline for river bank protection, BWDB, Table: 8.3)		
Ψ_{cr} =	Critical shear stress parameter	=	**0.035**
	(Guideline for river bank protection, BWDB, Table: 8.5)		
K_t =	Turbulence factor	=	**1.5**
	(Guideline for river bank protection, BWDB, Table: 8.4)		
K_h =	Depth factor, For non-developed profile: $K_h = (1+h/k_s)^{-0.2}$	=	0.54
	h = Water depth, {considering (HFL - Bed level)}	=	8.59 m
	k_s = Bed roughness given approximately by: $k_s = D_n$	=	**0.40** m
	h/k_s	=	21.49
θ =	Angle of repose, (Pilarczyk 2000)	=	**40** ^{0}
	(Guideline for river bank protection, BWDB, Table: 8.1)		
K_s =	$[1-(\sin\alpha/\sin\theta)^2]^{0.5}$ =[1-(Sin1.5/Sin40)^2)]^0.5	=	0.72
D_n =	Cubic dimension of CC Blocks	D_n =	**0.079 m**
		=	**79 mm**

Provided Size = 400 x 400 x 400 (60%)
Provided Size = 300 x 300 x 300 (40%)

As a result of the above calculation, the required minimum diameter D_n of CC blocks is 79 mm and required equivalent weight is 1 kg.

■ Economic Comparison of Block Types

As a result of comparing economy of various CC blocks ranging from the smallest size of 200×200×200 to the 400×400×400 class that can be manually dumped into the river from a barge, the 400×400×400 type was found to be the most cost effective.

As a result of holding discussions with the BWDB design department, since a random mixture of two types of CC blocks, i.e. 300×300×300 (62.1kg) and 400×400×400 (147.2kg), has so far been used (in anticipation of a good biting effect) in works on Manu River (adopting 400×400×400 blocks and 300×300×300 blocks in the ratio of 60% to 40%) without failure, a similar mix of the 300×300×300 and 400×400×400 sizes is planned here too.

Table 18 Economic Comparison Sheet of CC Blocks (per tK/100m^3)

CC Block Standard	200*200*200	250*250*250	300*300*300	400*400*400
Blocks/100m^3	12,500	6,410	3,704	1,563
Manufacturing unit cost (including materials) (tK/unit)	94.99	127.03	205.88	477.12
Installation (tK/unit)	1602	1602	1602	1602
Works cost (tK/unit)	1696.99	1729.03	1807.88	2079.12
Works cost (tk/100m^3)	21,212,375	11,083,082	6,696,388	3,249,665

(3) Design of Length of Launching apron

1) Examination of Length of Launching apron

According to the Guidelines for River Bank Protection (2010 BWDB), as is shown in the following figure, the length of foot protection work is stipulated as 1.5 times the scouring depth.
According to the previous calculation, the required scouring depth is 1.63m, therefor; this scouring depth should be adopted in examination of Length of Launching apron.
Examination of termination Length of Launching apron is used BWDB’s design experience of same total block volume of standard place. Therefore, block volume of termination place has set 7.0m^3 as same volume of standard place volume 7.0m^3

T (length of foot protection work) ＝1.5D

D: Scouring depth (m)

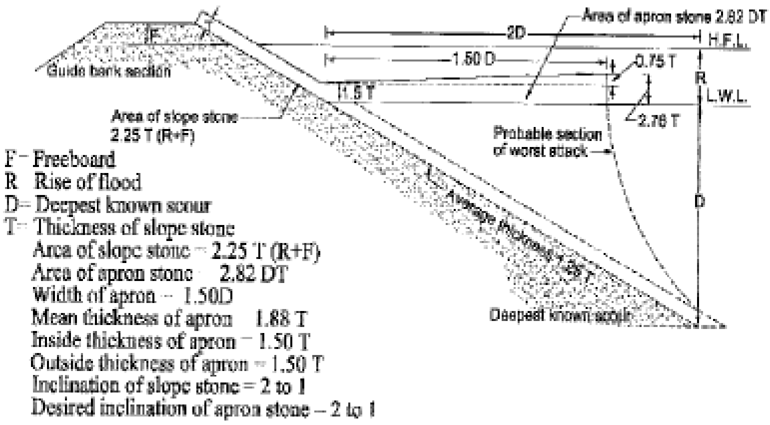


Fig.8.7c Shape of apron suggested by Rao(1946)

Figure 32 Guidelines for River Bank Protection (2010 BWDB), p162

■Calculation of Length of Launching apron

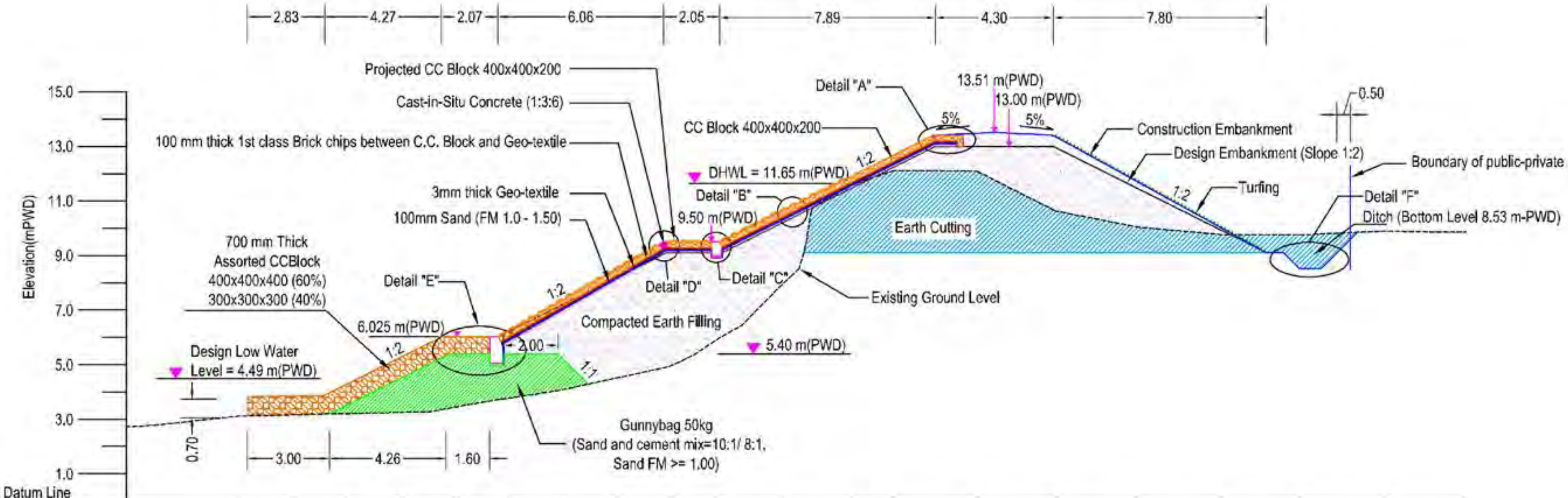
□Normal place

T（Length of Launching apron）＝1.5D＝1.5×1.63=2.45m≒3.0m

D：Scouring depth 1.63m

□Termination place

T（Length of Launching apron）＝7m^3/m÷0.7m=10.0m



Existing RL in m-PWD	2.790	3.120	3.230	3.210	3.250	3.460	3.780	4.100	4.530	4.950	5.960	7.880	11.576	12.120	12.045	10.630	10.040	9.770	9.760	9.890	9.850
Distance from LB in m	147.60	150.60	152.60	154.60	156.60	158.60	160.56	162.55	164.55	166.55	168.27	170.54	171.60	174.60	177.60	180.50	183.50	186.50	189.50	192.50	195.50

Cross Section No # 12

Figure 33 Pilot Project Standard Cross Section

Appendices

Design of Pilot Repair Works of Manu River Flood Control Embankment at Noarai,Akhailkura in Moulvibazar District

Calculation Report

May2015

BWDB Design Circle

JICA Expert Team

Table of Contents

1.1. Slip Circle analysis

Examination of Circular Slip

The Bishop method using the set soil constants is used to examine circular slip. As indicated in the following table, the results of examination show that the safety factor is secured and there is no issue with safety in the design cross section.

$$F_s = \frac{R\Sigma(\{c.l\cos\alpha + (1+Kv)W'\tan\varphi\}/m\alpha)}{\Sigma\ \{(1+Kv)W\cdot x + Kh\cdot W\cdot y + P\cdot a\}}$$

$$m_\alpha = \cos\alpha + \frac{\tan\varphi'\cdot\sin\alpha}{Fs}$$

Fs: Safety factor
C: Adhesive force of soil (KN/m2)
φ: Internal angle of friction of soil (°)
l: Length of slice circle (m)
W': Effective weight of slice (KN/m)
W: Weight of slice (KN/m)
Kh: Design horizontal seismic coefficient (not considered in this case)
Kv: Design vertical seismic coefficient (not considered in this case)
R: Radius of circle (m)
α: Angle of inclination of the slice base (°)
x: Horizontal distance between the slice center of gravity and center of circle (m)
y: Vertical distance between the slice center of gravity and center of slip circle (m)
P: Water pressure acting on earth clump inside the slip circle (KN/m)
α: Vertical distance of horizontal line passing through the pressure point and center of the slip circle (m)

Table Calculation Cases and Results

Calculation case	Frontal water level	Safety factor	
		River side	Country side
H.W.L.	11.65mPWD	2.56>1.50 OK	1.80>1.50 OK
Flood	9.10mPWD	1.95>1.50 OK	1.80>1.50 OK
L.W.L.	4.49mPWD	1.76>1.20 OK	1.81>1.20 OK

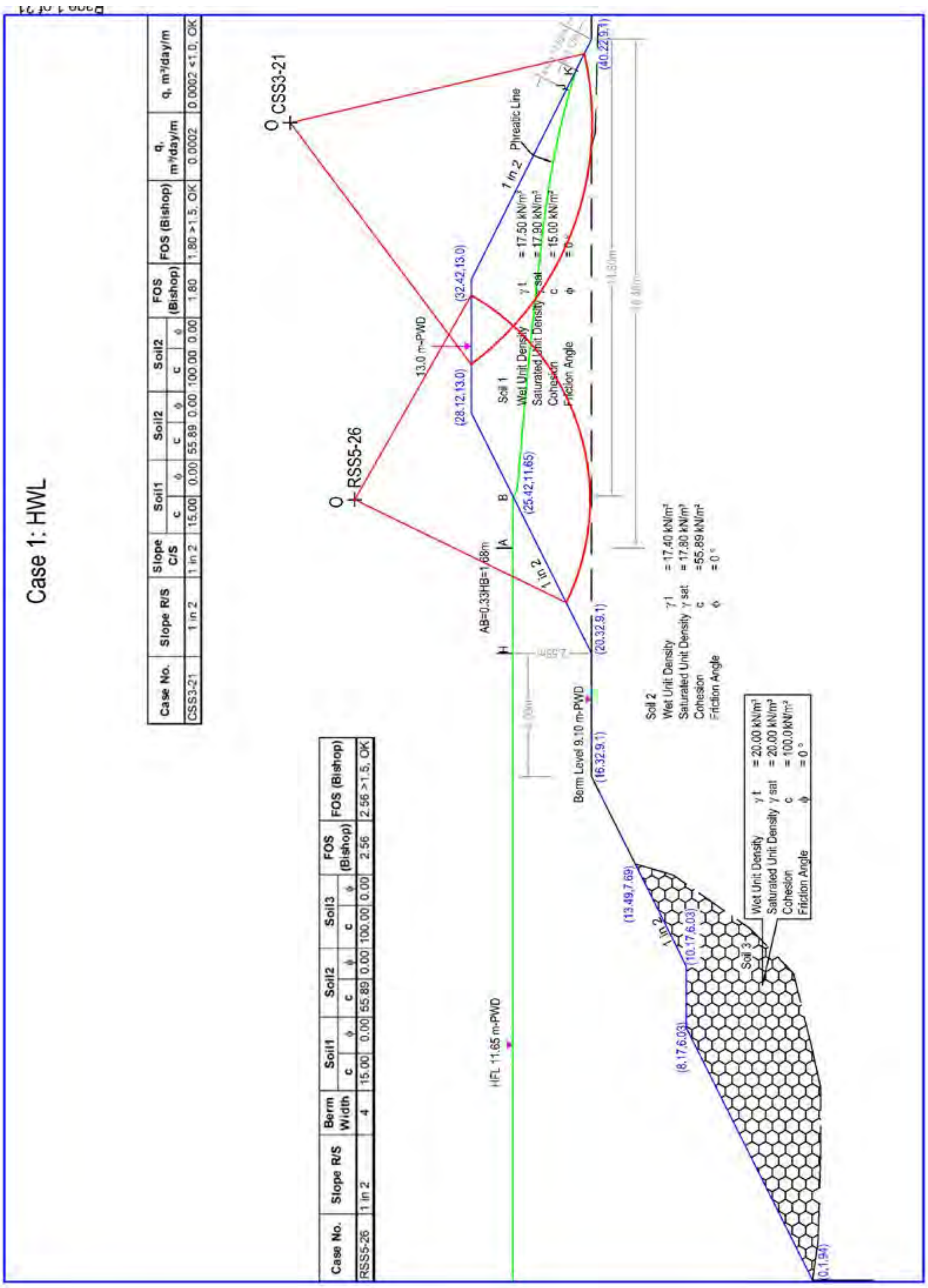


Figure Calculation Case 1: Frontal water level at H.W.L. 11.65mPWD

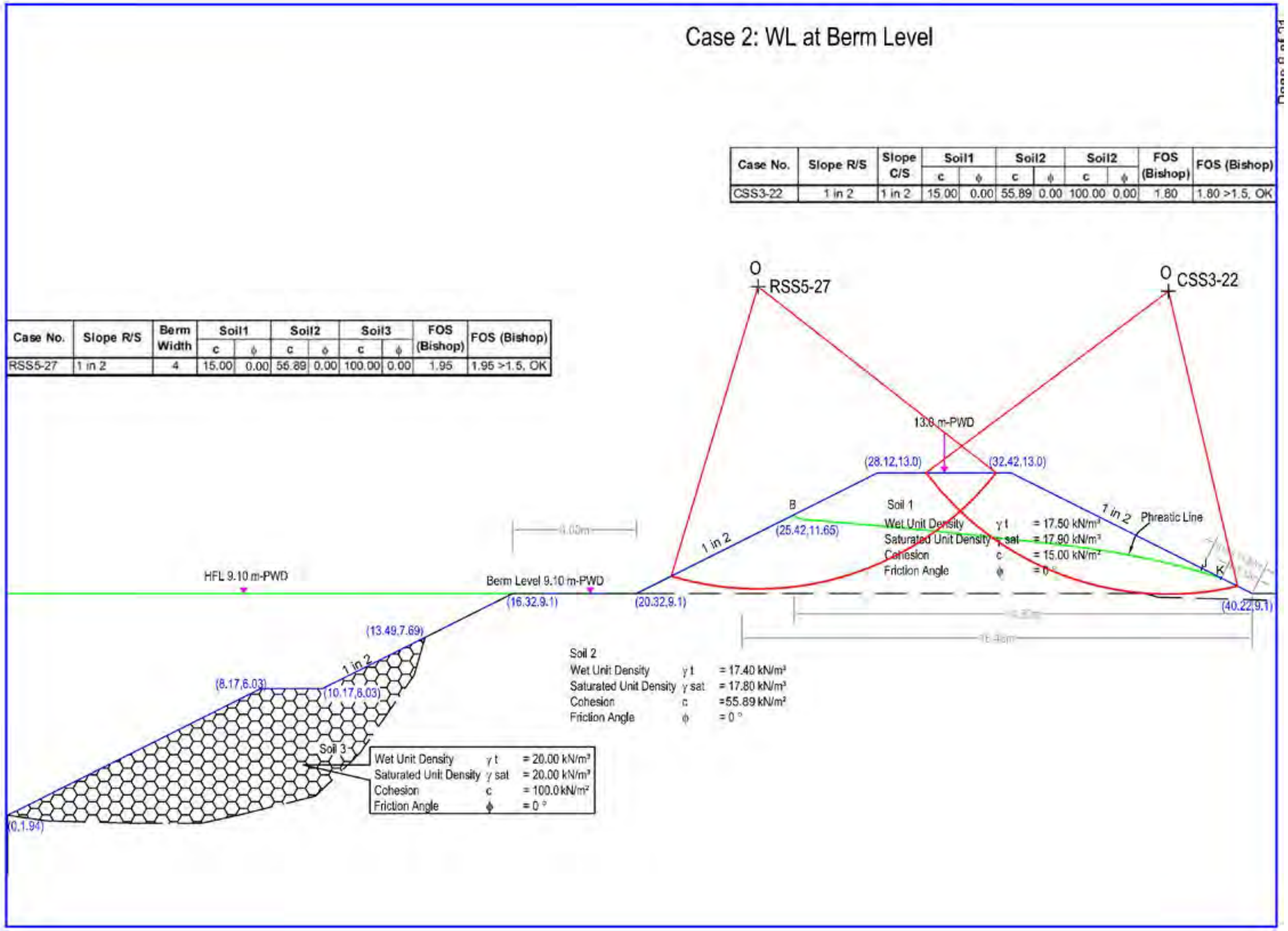
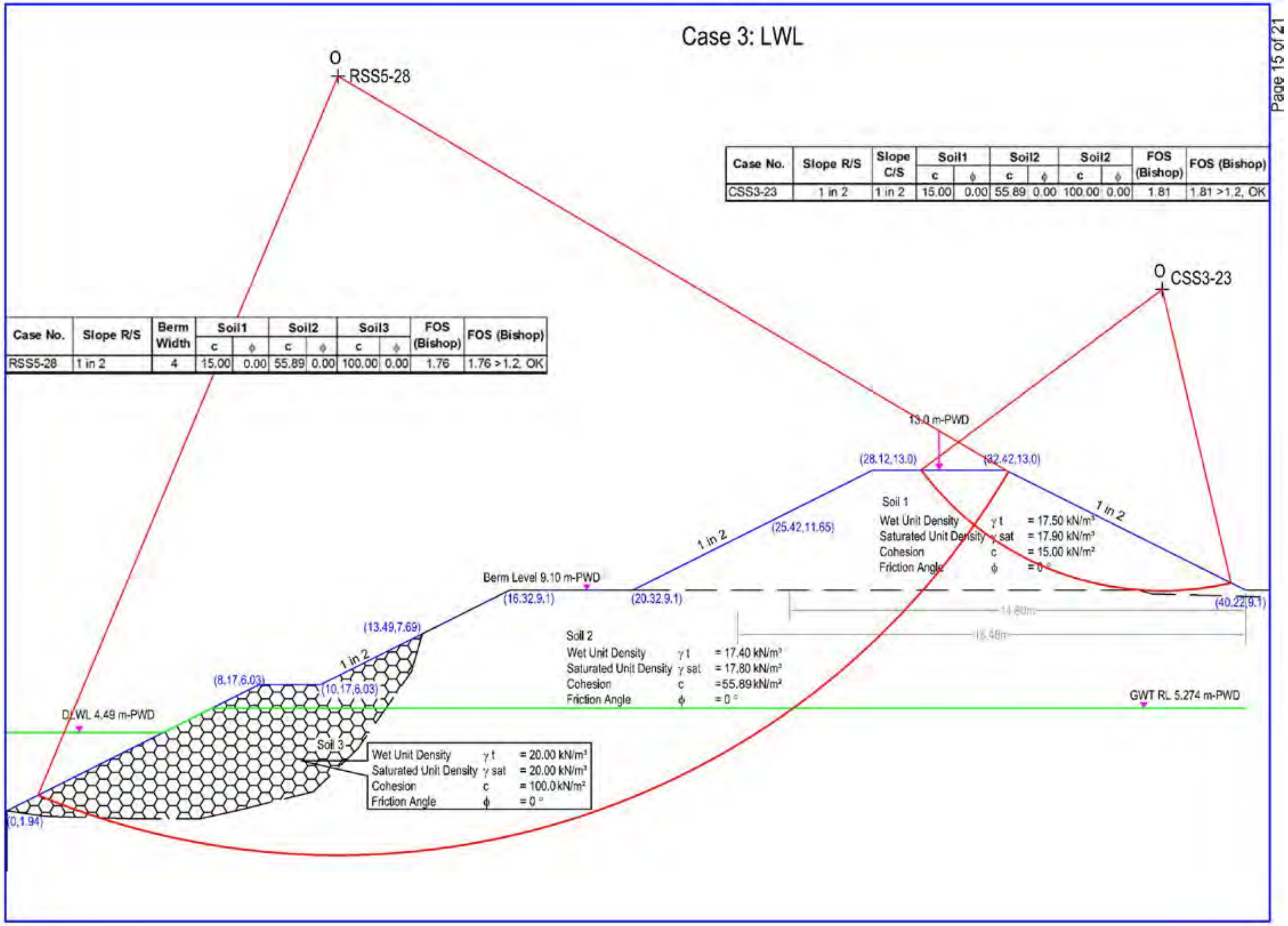


Figure Calculation Case 2: Frontal water level at 9.1mPWD



Case No.	Slope R/S	Berm Width	Soil1		Soil2		Soil3		FOS (Bishop)	FOS (Bishop)
			c	ϕ	c	ϕ	c	ϕ		
RSS5-28	1 in 2	4	15.00	0.00	55.89	0.00	100.00	0.00	1.76	1.76 >1.2, OK

Case No.	Slope R/S	Slope C/S	Soil1		Soil2		Soil2		FOS (Bishop)	FOS (Bishop)
			c	ϕ	c	ϕ	c	ϕ		
CSS3-23	1 in 2	1 in 2	15.00	0.00	55.89	0.00	100.00	0.00	1.81	1.81 >1.2, OK

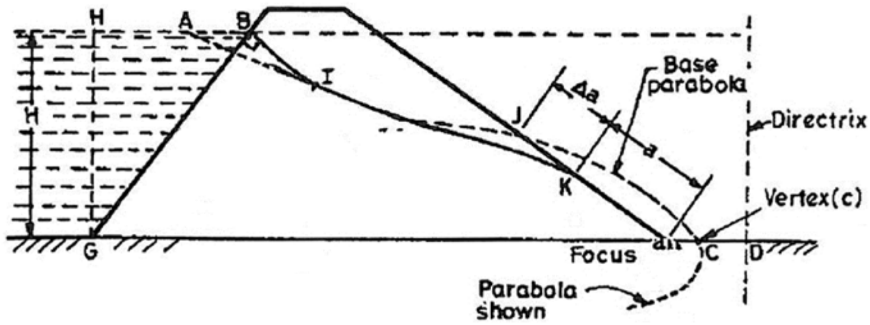
Figure Calculation Case 3: Frontal water level at D.L.W.L. 4.49mPWD

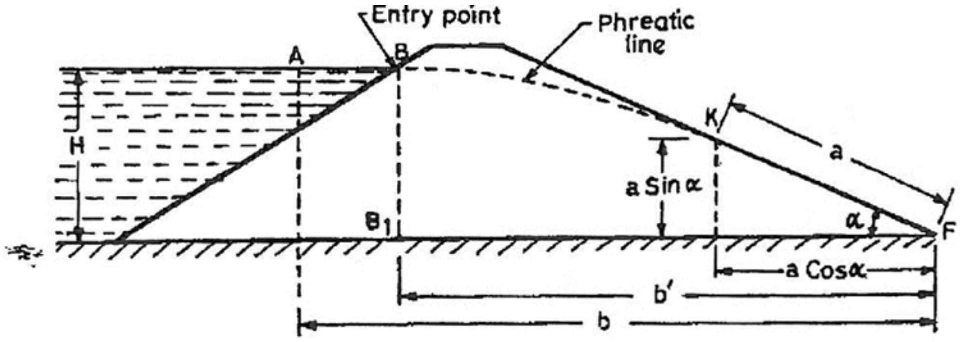
1.2. Seepage

Seepage Flow Analysis

The Casagrande method is used to examine the seepage line. The following sections describe the concept and results of seepage line examination by the Casagrande method.

As a result, the seepage line appears in the embankment slope, however, since it is planned to use clay with very low permeability as the embankment material, there should be no seepage failure.





$\Delta a = (a+\Delta a)\ \frac{180° - \alpha}{400°}$

When a is 30°:

$$a = \frac{b'}{\cos\alpha} - \sqrt{\left(\frac{b'}{\cos\alpha}\right)^2 - \left(\frac{H}{\sin\alpha}\right)^2}$$

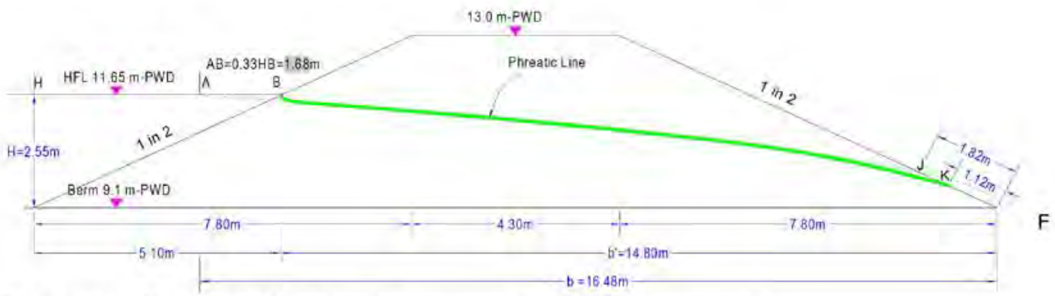
When a is30°~ 60°:

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2 cot^2\alpha}$$

Q = k ・S (**k**: coefficient of permeability)

S = $\sqrt{b^2 + H^2} - b$

Phreatic Line



The equation of base parabola is given by $\sqrt{x^2 + y^2} = x + S$

AB ≈ 0.33 HB =0.33 * 5.10 = 1.68

x =	**16.48**
y =	**2.55**

$$S = \sqrt{x^2 + y^2} - x$$

S = **0.196**

A few more coordinate of base parabola at known distance (x) are orked out in the following table using,

$$y = \sqrt{S^2 + 2xS}$$

x	y² = 2xS+S² = 0.392x + .154	y
0.00	0.038	0.196
-0.07	0.011	0.105
0.00	0.038	0.196
0.50	0.235	0.484
1.00	0.431	0.656
2.00	0.823	0.907
3.00	1.215	1.102
4.00	1.607	1.268
5.00	2.000	1.414
6.00	2.392	1.547
7.00	2.784	1.669
8.00	3.176	1.782
9.00	3.569	1.889
10.00	3.961	1.990
11.00	4.353	2.086
12.00	4.745	2.178
13.00	5.138	2.267
14.00	5.530	2.352
15.00	5.922	2.434
16.00	6.314	2.513
16.48	6.503	2.550

At entry, the phreatic line is started from the point B in such a way that it becomes at right angles to the u/s face of the embankment. A reverse curvature is, given as shown in above figure. At exit, the point K at which the phreatic line intersects the d/s face can be easily obtained by using

$$\Delta a = (a + \Delta a)\left[\frac{180 - \alpha}{400}\right]$$

Where, $\tan \alpha = \frac{1}{2}, \; or \; \alpha = 26^{0}.54$

$(a + \Delta a) =$ Distance FJ, i.e; the distance of the focus form the point at which the base parabola intersects the d/s face

= **1.82**

$\alpha =$ 26.57 Slope 1: **2.0**

$\Delta a =$ 1.82*((180-26.57)/400) = 0.70

a= 1.82-0.70 = **1.12**

1.3. Consolidation Settlement

Examination of Consolidation Subsidence

In the Design Manual, the Terzaghi formula is used to examine consolidation subsidence. In conducting examination, the set soil constants are used. As a result, since subsidence of 39 centimeters is confirmed, extra banking of 39cm ≒ 40.0cm will be added to compensate.

$$S = \frac{C_c \cdot H}{1+e_0} \log_{10} \frac{P_0 + \Delta P}{P_0}$$

S: Amount of subsidence of single layer

Cc: Consolidation index

H: Compaction layer thickness

e_o: Initial void ratio

Po: Effective overburden stress before embankment construction

⊿P: Increase in overburden stress after embankment construction

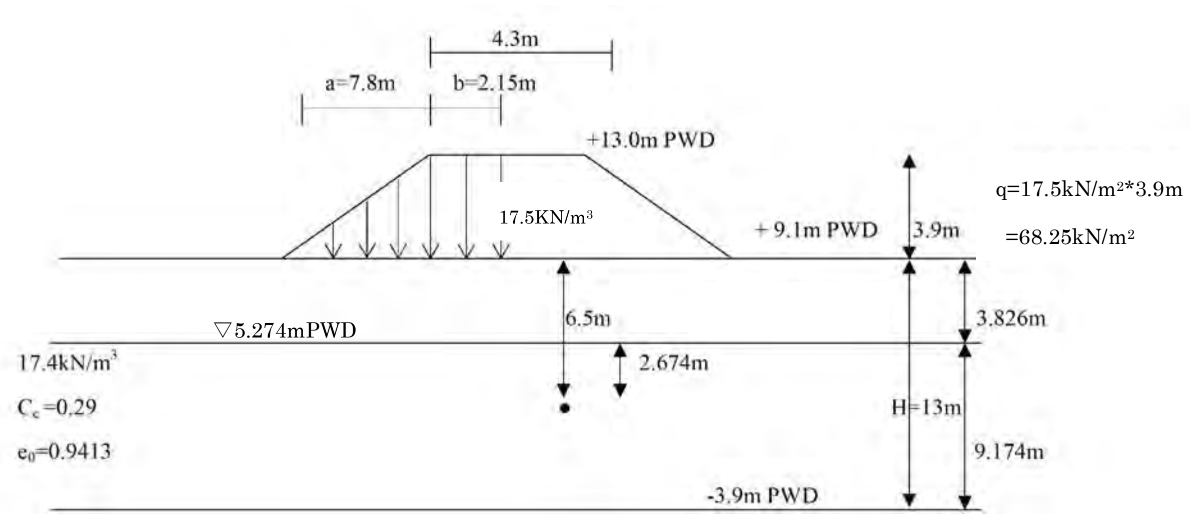


Figure Model Diagram of Consolidation Calculation

$$P_0 = 17.4kN/m^3 * 3.826m + (17.4 - 9.81)kN/m^3 * 2.674$$
$$= 86.87kN/m^2$$

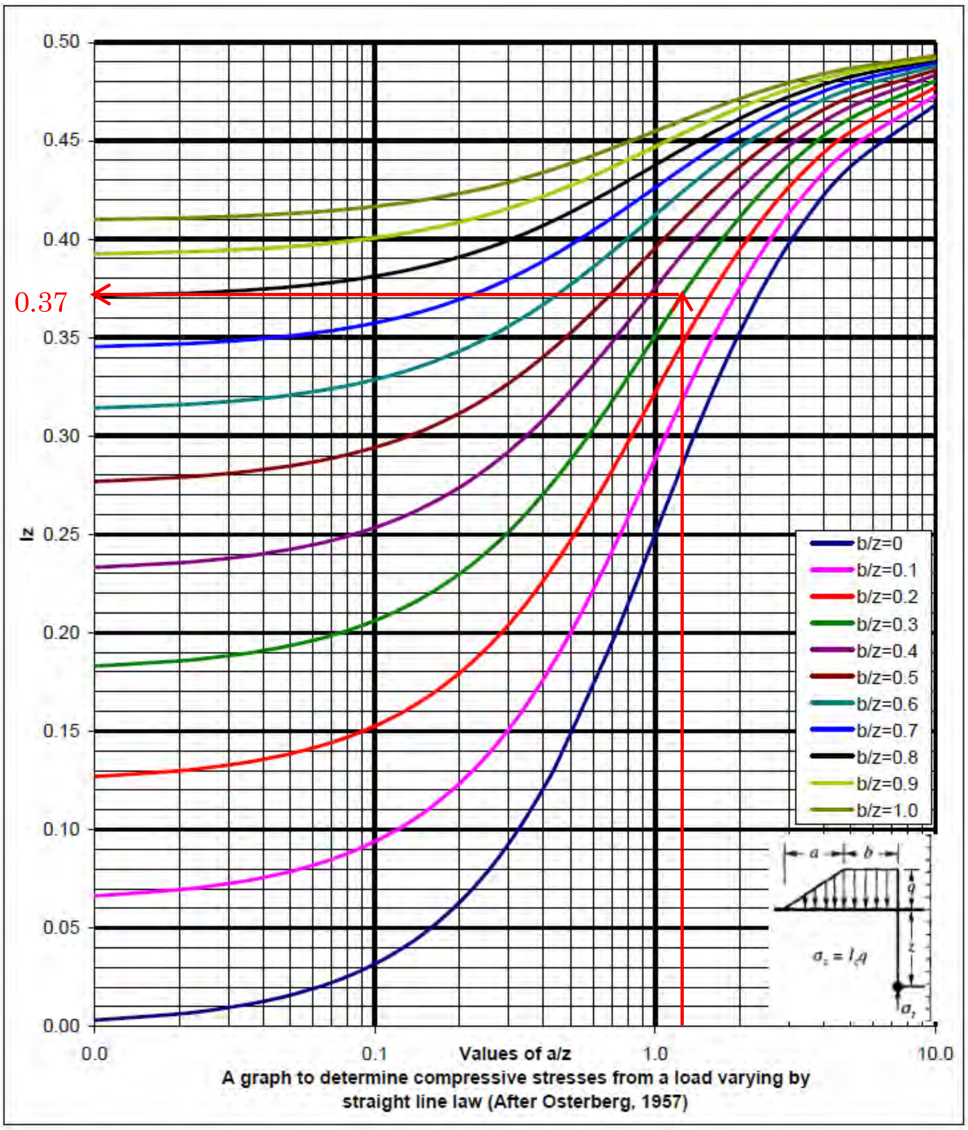


Figure Relationship between Increased Stress and Depth (Osterberg,1957)

$$\frac{a}{z} = \frac{7.8m}{6.5m} = 1.2$$

$$\frac{b}{z} = \frac{2.15m}{6.5m} = 0.33$$

According to the Osterberg figure, Iz=0.37

Concerning Iz at depth of 6.5 meters, since load is considered on the left and right sides of the embankment, Total Iz = 0.37*2=0.74.

$$\Delta P = I_z \times g = 0.74 \times 68.25 KN/m^2 = 50.51 KN/m^2$$

$$S = \frac{C_c H}{1+e_0} \log\left(\frac{P_0 + \Delta P}{P_0}\right)$$

S: Amount of subsidence of single layer

Cc: Consolidation index = 0.29 (result of indoor testing)

H: Compaction layer thickness = 13m

e_o: Initial void ratio = 0.9413 (result of indoor testing)

Po: Effective overburden stress before embankment construction

$= 86.87 \text{KN/m}^2$

⊿P: Increase in overburden stress after embankment construction

$= 49.64 \text{KN/m}^2$

$$S = \frac{0.29 * 13}{1 + 0.9413} \log\left(\frac{86.87 + 50.51}{86.87}\right)$$

$$= 1.94 * 0.199$$

$$= 0.39m$$

Therefore, the extra banking shall be 0.39m = 0.4m.

2.1. CC block for slope protection

Design of Bank Protective Works
at Pilot Project Site along Right Bank of Manu River

Target Cross Secttion No. 16

Design data:

1. Highest flood level				=	11.65 m-PWD
2. Free board				=	0.90 m
3. Crest level (Top level)				=	12.55 m-PWD
				≈	13.00 m-PWD
4. Berm level				=	9.10 m-PWD
5. Design Low Water Level				=	4.49 m-PWD
6. Average river bed level				=	3.056 m-PWD
7. Revetment material : C. C. Block & Gabions filling by stone					
8. Bank slope above berm level, α	= 1V:2H	= 1: 2.0	α	=	26.57^{0}
9. Bank slope below berm level, α	= 1V:2H	= 1: 2.0	α	=	26.57^{0}
10. Scour slope below bed level, α	= 1V:2H	= 1: 2.0	α	=	26.57^{0}

Design against current:

Pilarczyk Equation:

$$D_n \geq \frac{0.035 * u^2}{\Delta_m * 2g} * \frac{\phi_{sc} K_\tau K_h}{K_s * \psi_{cr}}$$

(BWDB, Guideline for River Bank Protection, May 2010, Page- 142)

(1) CC blocks, concrete with stone aggregate (single-layer above construction water level)

D_n =	Cubic dimension of CC Blocks		
Δ_m =	Relative density of submerged material = $(\rho_s - \rho_w) / \rho_w$	=	1.30
ρ_s =	Density of CC block	=	**2250** kg/m³
ρ_w =	Density of water	=	980 kg/m³
u =	Flow velocity	=	**1.50** m/sec
g =	Acceleration due to gravity	=	9.81 m/s²
ϕ_{sc} =	Stability factor for current (Continuous protection)	=	**0.65**
	(Guideline for river bank protection, BWDB, Table: 8.3)		
Ψ_{cr} =	Critical shear stress parameter	=	**0.05**
	(Guideline for river bank protection, BWDB, Table: 8.5)		
K_τ =	Turbulence factor	=	**1.5**
	(Guideline for river bank protection, BWDB, Table: 8.4)		
K_h =	Depth factor, For non-developed profile: $K_h = (1+h/k_s)^{-0.2}$	=	0.51
	h = Water depth, {considering (HFL - Design Low Water Level)}	=	7.16 m
	k_s = Bed roughness given approximately by: $k_s = D_n$	=	**0.25** m
	h/k_s	=	28.64
θ =	Angle of repose, (Pilarczyk 2000)	=	**40** ^{0}
	(Guideline for river bank protection, BWDB, Table: 8.1)		
K_s =	$[1-(\sin\alpha/\sin\theta)^2]^{0.5}$ =[1-(Sin26.57/Sin40)^2)]^0.5	=	0.72
D_n =	Cubic dimension of CC Blocks **D_n**	=	**0.043 m**
		=	**43 mm**

Provided Size = 400 x 400 x 200

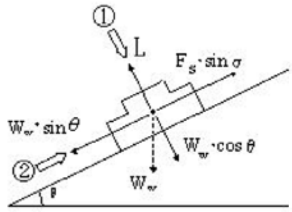
2.2. Projected CC block for slope protection

Stability should be confirmed with the following formula.

$$\mu\,(W_w \cdot \cos\theta - L) \geqq ((W_w \cdot \sin\theta)^2 + D^2)^{1/2}$$

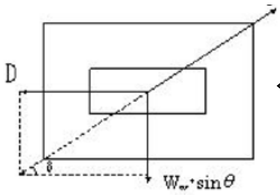
$$L = \rho_w/2 \cdot C_L \cdot A_b \cdot V_d^2\ (\mathrm{kgf})\ 【\mathrm{N}】$$

$$D = \rho_w/2 \cdot C_D \cdot A_D \cdot V_d^2\ (\mathrm{kgf})\ 【\mathrm{N}】$$

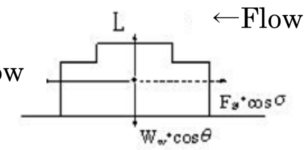


This shows the condition of a block placed on the right bank

Force viewed from the direction of ①



Force viewed from the direction of ②



Slope maximum in cline direction

【Design conditions】

Design flow velocity: 1.50(m/s)

H.W.L.: 11.65mPWD

Slope gradient: 1:2.0

Design cross section: Measurement point No.16

Deepest riverbed elevation: 1.68mPWD

Design water depth: H.W.L.11.65mPWD - Deepest riverbed elevation 1.68mPWD = 9.97m

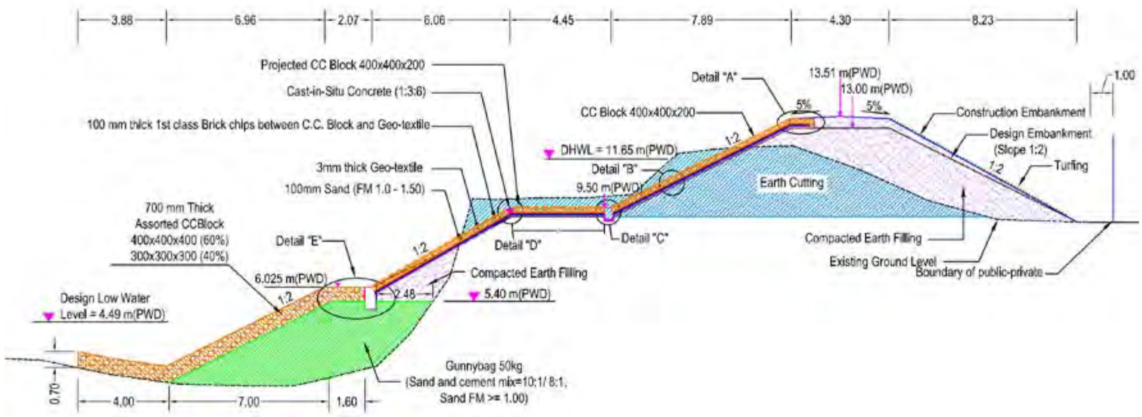


Figure Design cross section: Measurement point No.16

μ : Coefficient of friction μ=0.65 (coefficient of friction between soil and suction prevention material)

W_w : Weight of blocks in water (kg)

θ : Slope gradient; 1:2.0

ρ_b : Density of blocks; 2250(kg/m^3)

ρ_w : Density of water ρ_w; 980 (kg/m^3)

g : Acceleration of gravity; 9.81(m/s^2)

A_b : Projected area of blocks when viewed from above; 0.16(m^2)

A_D : Projected area of blocks pertaining to drag; 0.09 (m^2)

t_b : Projected area of blocks pertaining to drag; 0.2(m)

C_L : Uplift coefficient of blocks; 0.15

C_D : Uplift coefficient of blocks; 0.70

V_d : Design flow velocity at level t_b (m/s)

$$V_d = \frac{8.5+5.75 log10\left(\frac{tb}{ks}\right)+2}{6.0+5.75\log10\left(\frac{Hd}{ks}\right)} \cdot V_o(\text{m/s})$$

Hd: Design depth of water; 9.97(m)

ks: Equivalent roughness; 0.08

Vo: Design flow velocity Vo=1.5(m/s)

The Formula for V_d is as follows

$$V_d = \frac{8.5 + 5.75\log_{10}\left(t_b/k_s\right) + 2}{6.0 + 5.75\log_{10}\left(H_d/k_s\right)} \times Vo$$

$$= \frac{8.5 + 5.75\log_{10}\left(0.2/0.08\right) + 2}{6.0 + 5.75\log_{10}\left(9.97/0.08\right)} \times 1.5$$

=1.06(m/s)

The Lift L and Drag D acting on the CC blocks with projections are sought.

$$L = \frac{1}{2} \cdot \rho_w \cdot C_L \cdot A_b \cdot V_d{}^2$$

$$= \frac{1}{2} * 980 * 0.15 * 0.16 * 1.06^2$$

=13.21(N)

$$D = \frac{1}{2} \cdot \rho_w \cdot C_D \cdot A_D \cdot V_d{}^2$$

$$= \frac{1}{2} * 980 * 0.7 * 0.09 * 1.06^2$$

=34.69(N)

The underwater weight of Projected CC blocks and 20cm thickness is sought.

$$W_w \geq \frac{\mu^2 L + \sqrt{\mu^4 L^2 - (\mu^2 - \tan^2\theta)(\mu^2 L^2 - D^2)}}{\cos\theta(\mu^2 - \tan^2\theta)}$$ =133.61(N)=13.63(kg)

The air weight of Projected CC blocks and 20cm thickness is sought.

$$W = \left(\frac{\rho_b}{\rho_b - \rho_w} \right) * W_w$$ =232.81(N) =23.76(kg)

If the product weight of projected CC blocks and 20cm thickness is the same or greater than the required weight in air, the blocks will be stable with request to the flow velocity.

i.e.: W (76.5kg)>23.76kg・・・・ OK

2.3. CC block for foot protection

Design of Foot Protection Work

In examining the foot protection blocks, the Bangladesh Guidelines for River Bank Protection (2010 BWDB), p142 will be followed.

$$D_n \geq \frac{0.035 * u^2}{\Delta_m * 2g} * \frac{\phi_{sc} K_\tau K_h}{K_s * \psi_{cr}} \quad \text{(Source: Guideline p142)}$$

Where,

D_n	(m)	Equivalent diameter (cover layer)
Δ_m	(-)	$(\rho_s-\rho_w)/\rho_w$ = relative density of submerged material
ρ_s	(kg/m³)	Density of protection material
ρ_w	(kg/m³)	Density of water
$\bar{u}$	(m/s)	Depth averaged mean flow velocity
g	(m/s²)	Acceleration due to gravity
ϕ_{sc}	(-)	Stability factor for current
K_τ	(-)	Turbulence factor
K_h	(-)	Depth factor, dependent on the assumed velocity profile and water depth (h) to equivalent roughness height ratio
K_s	(-)	Slope parameter
Ψ_{cr}	(-)	Critical shield's parameter
α	(°)	Slope angle of bank or structure
θ	(°)	Angle of repose considering the material specific internal friction

Design of Bank Protective Works
at Pilot Project Site along Right Bank of Manu River

Target Cross Secttion No. 16

Design data:

1.	Highest flood level					=	11.65	m-PWD
2.	Free board					=	0.90	m
3.	Crest level (Top level)					=	12.55	m-PWD
						≈	13.00	m-PWD
4.	Berm level					=	9.10	m-PWD
5.	Design Low Water Level					=	4.49	m-PWD
6.	Average river bed level					=	3.056	m-PWD
7.	Revetment material :	C. C. Block & Gabions filling by stone						
8.	Bank slope above berm level, α	= 1V:2H	=	1: 2.0	α	=	26.57 ^{0}	
9.	Bank slope below berm level, α	= 1V:2H	=	1: 2.0	α	=	26.57 ^{0}	
10.	Scour slope below bed level, α	= 1V:2H	=	1: 2.0	α	=	26.57 ^{0}	

CC blocks, concrete with stone aggregate (dumping below construction water level)

D_n =	Cubic dimension of CC Blocks			
Δ_m =	Relative density of submerged material = $(\rho_s - \rho_w) / \rho_w$	=	1.30	
ρ_s =	Density of CC block	=	**2250**	kg/m³
ρ_w =	Density of water	=	980	kg/m³
u =	Flow velocity	=	**1.50**	m/sec
g =	Acceleration due to gravity	=	9.81	m/s²
$Ø_{sc}$ =	Stability factor for current (Continuous protection)	=	**0.8**	
	(Guideline for river bank protection, BWDB, Table: 8.3)			
Ψ_{cr} =	Critical shear stress parameter	=	**0.035**	
	(Guideline for river bank protection, BWDB, Table: 8.5)			
K_τ =	Turbulence factor	=	**1.5**	
	(Guideline for river bank protection, BWDB, Table: 8.4)			
K_h =	Depth factor, For non-developed profile: $K_h = (1+h/k_s)^{-0.2}$	=	0.54	
	h = Water depth, {considering (HFL - Bed level)}	=	8.59	m
	k_s = Bed roughness given approximately by: $k_s = D_n$	=	**0.40**	m
	h/k_s	=	21.49	
θ =	Angle of repose, (Pilarczyk 2000)	=	**40**	°
	(Guideline for river bank protection, BWDB, Table: 8.1)			
K_s =	$[1-(\sin\alpha/\sin\theta)^2]^{0.5}$ =[1-(Sin1.5/Sin40)^2)]^0.5	=	0.72	
D_n =	Cubic dimension of CC Blocks	**D_n =**	**0.079**	**m**
		=	**79**	**mm**
	Provided Size = 400 x 400 x 400 (60%)			
	Provided Size = 300 x 300 x 300 (40%)			

As a result of the above calculation, the required minimum diameter D_n of CC blocks is -79 millimeters.

2.4. Length of launching apron

Design of Bank Protective Works
at Pilot Project Site along Right Bank of Manu River

Modarate bend

Design data

Maximum Discharge at Pilot Project Site (20 year return period)		=	850 m^3/sec
Highest flood level at Pilot Project Site (20 year return period)		=	11.65 m-PWD
Free board		=	0.9 m
Crest level (Top level)		=	12.55 m-PWD
	Say	=	**13.00 m-PWD**
Low Water Level		=	**4.49** m-PWD
Bed level		=	**2.50** m-PWD
Average diameter of river bed material, d_{50}		=	0.15 mm

Thickness of Slope pitching

The thickness of the pitching on the river side may be calculated by:

1. By Inglish (1949)	(For 850 m^3/sec)	$t = 0.06\ Q^{1/3}$	=	0.57 m
2. By Gales (1938)	(For 0.03 million cusec)	t = -	= -	m
3. By Spring (1903)	(For slope about 9 inch per mile)	t = 37" *1.0	=	0.94 m
		Let, thickness of pitching, t	**=**	**0.20 m**

Launching apron

Thickness of Launching apron

Scour slope = 2:1

Thickness of launched apron = **1.25** t, Where t is thickness of pitching

Volume of stone required in the launched apron = $\sqrt{(2^2+1^2)}$*D* 1.25*t
= (1.25*t)*√5*D

Width of unlaunched apron = 1.5*D

Thickness of the unlaunched apron, T = (1.25*t)*√5*D/(1.5*D)= 1.86 *t ; Say 1.9 *t

Hence $\boxed{\mathbf{T = 1.9*t}}$ = T = 0.38 m

; Say **T** = **0.70 m**

Scour

Lacey's regime formula is widely used to find out scour depth in alluvial rivers.

f= Lacey's silt factor =$1.76(d_{50})^{(1/2)}$	=1.76*(0.15)^(1/2)	=	0.68
Lacey's normal scour depth, R = $0.47(Q/f)^{1/3}$=	0.47*(850/0.68)^(1/3)	=	**5.06 m**
Multyplying factor "X" for Modarate bend		=	1.50
Lacey's factored scour depth, X*R = 1.50 * 5.06		=	**7.59 m**
Lacey's factored scour level,		=	**4.06 m-PWD**
h = Depth of flow, may be calculated as (HFL-LWL)		=	7.16 m
Scour depth at design discharge , Ds = XR - h		=	0.43 m
Bed level depth below LWL , Ds = LWL-Bed level		=	1.99 m
Length of Launching apron = 1.5 Ds		=	2.99
Lengh of apron	; Say	=	**3.00 m**

Bangladesh Water Development Agency
The People's Republic of Bangladesh

THE PROJECT FOR CAPACITY DEVELOPMENT OF MANAGEMENT FOR SUSTAINABLE WATER RELATED INFRASTRUCTURE IN THE PEOPLE'S REPUBLIC OF BANGLADESH

Assessment on Earthquake Resistance of the Embankment

Appendix

Of

River Embankment Design Manual in Bangladesh

July 2015

JICA EXPERT TEAM

CONTENTS

1. Objective

The purposes of this report are specifying the method for assessing slope stability of the river embankment in Bangladesh during an earthquake and accordingly assessing the slope stability of the embankment at Bogra which is dominantly sand in both the embankment and its associated underground.

This report is the appendix of *River Embankment Design Manual in Bangladesh*. This report was prepared in accordance with *the Design Manual*

In this report, followings are addressed

(1) Assessment Method of the Slope Stability of the Embankment in Bangladesh during an Earthquake
(2) Case Study of the Slope Stability Assessment of the Embankment during an Earthquake

2. Assessment Method of the Slope Stability of the Embankments in Bangladesh during an Earthquake

2.1 Flow of Assessment on the Slope Stability of the Embankments

Assessment of the slope stability during an earthquake is conducted in accordance with following entire flow described in Fig 2.1.1

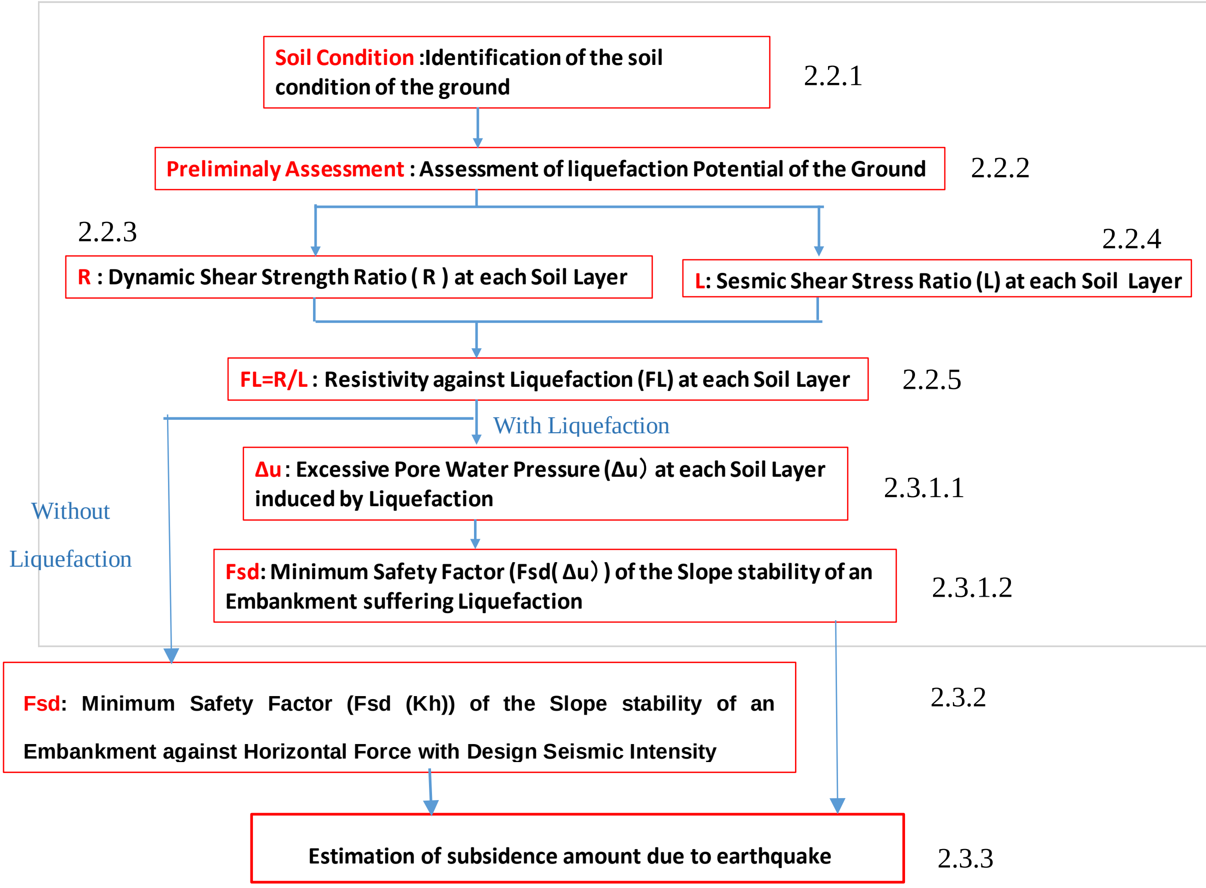


Fig 2.1.1 Assessment Flow of the Slope Stability during an Earthquake

Each item of assessment flow is described in the following sections.

2.2 Assessment of Liquefaction Potential of the ground

2.2.1 Identification of the soil condition of the ground

Following parameters of the underground of the embankment and design seismic intensity are put together based on the soil investigation and the seismic intensity in Bangladesh.

- X: Depth from the ground surface (m)
- h_w: Ground water level from the ground surface (m)
- FC: Fine particle content (%)
- N: N value of the ground
- ρ_{t1}: Density of soil in shallower than water table position (kN/m^3)
- ρ_{t2}: Density of soil in deeper than the water table position. (kN/m^3)
- PI: plasticity index
- Ks: Design seismic intensity at the embankment site.

2.2.2 Assessment of Liquefaction Potential of the Ground

Evaluation on liquefaction potential is conducted for the soil layer existing in 20 m or less below the ground surface based on the following 2 criterions as a preliminary screening. When all of these 2 criterions are satisfied, the ground is evaluated as having the potential of suffering liquefaction.

- The ground water level is 10 m or less below the ground surface.
- The soil layer has FC (fine particle content) of not higher than 35%, or even if it is more than 35%, the plasticity index is 15 or less.

2.2.3 Dynamic Shear Strength Ratio (R) at each Soil Layer

Dynamic shear strength ratio (R) at each layer is calculated from following equations. Here, R stands for a *Resistance Capability Ratio* against Liquefaction of each particular soil layer.

$$R = C_w \cdot R_L$$

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} & (N_a \geqq 14) \end{cases}$$

(in case of Sandy Soil)

$$N_a = C_1 \cdot N_1 + C_2$$

$$N_1 = 1.7 \cdot N/((\sigma'_v/98.0) + 0.7)$$

$$C_1 = \begin{cases} 1 & (0\% \leqq FC < 10\%) \\ (FC+40)/50 & (10\% \leqq FC < 60\%) \\ FC/20-1 & (60\% \leqq FC) \end{cases}$$

$$C_2 = \begin{cases} 0 & (0\% \leqq FC < 10\% \\ (FC-10)/18 & (10\% \leqq FC) \end{cases}$$

Where; C_w; The correction factor due to seismic motion characteristics

$(C_w = 1.0)$

R_L; Repeat 3-axis intensity ratio

N_1; N value equivalent to Effective stress 98.0 kN/m^2

N_a; Correction factor that takes into account the effect of particle size

C_1, C_2 ; Correction factor of N values by the fine fraction content

FC; Fine particle content (%)

(Passing mass percentage of particle below 75μm)

2.2.4 Seismic Shear Stress Ratio (L) at each Soil Layer

Seismic shear stress ratio (L) at each layer is calculated from following equations. Here, L stands for a *Induced Force Ratio* acting on each particular soil layer during an particular earthquake.

$$L = r_d \cdot K_s \cdot \sigma_v / \sigma_v'$$

$$r_d = 1.0 - 0.015\,x$$

$$\sigma_v = \rho_{t1} h_w + \rho_{t2}(x - h_w)$$

$$\sigma_v' = \rho_{t1} h_w + (\rho_{t2} - \gamma_w)(x - h_w)$$

Where; r_d; Reduction factor in the direction of the depth of the earthquake shear stress ratio

K_s; Design seismic intensity for liquefaction

x; Depth from the ground surface (m)

ρ_{t1}; Density of soil in shallow than water table position (kN/m^3)

ρ_{t2}; Density of soil in deeper than the water table position (kN/m^3)

γ_w ; Density of water (kN/m^3)

h_w; Depth from the ground surface (m)

σ_v : Total stress (kN/m^2)

σ'_v : Effective stress (kN/m^2)

2.2.5 Resistivity against Liquefaction (FL) at each Soil Layer

Resistivity against liquefaction (FL) at each layer is calculated from following equation. Here, FL stands for R/L identical to safety factor against complete liquefaction status at each particular soil layer during an particular earthquake.

FL=R/L

2.3 Estimation of subsidence Amount (Maximum) of an Embankment

2.3.1 Estimation of Minimum Safety Factor (Fsd(Δu)) of an Embankment suffering Liquefaction

2.3.1.1 Excessive Pore Water Pressure (Δ u) at each Soil Layer induced by Liquefaction

Excessive pore water pressure (Δ u) at each soil layer induced by liquefaction is calculated from following equation.

$$\Delta u = \sigma_v' \qquad \text{(in case of } F_L \leqq 1.0\text{)}$$
$$\sigma_v' \cdot F_L^{-7} \qquad \text{(in case of } F_L > 1.0\text{)}$$

Here, FL≦1.0 indicates that the relevant soil layer is liquefied losing all effective stress . FL>1.0 indicates that the relevant soil layer is partially liquefied, in other words, effective stress is partially reduced.

2.3.1.2 Minimum Safety Factor (Fsd(Δu)) of the Slope Stability of an Embankment suffering Liquefaction

With this excessive pore water pressure, minimum safety factor (Fsd(Δ u)) of the slope stability of the embankment suffering liquefaction is calculated from following equation.

$$F_{sd}(\Delta u) = \frac{\Sigma\left[c \cdot l + (W - u \cdot b - \Delta u \cdot b) \cdot \cos\alpha \cdot \tan\emptyset\right]}{\Sigma W \cdot \sin\alpha}$$

Where; $F_{sd}(\Delta u)$; Minimum safety factor that takes into account excess pore water pressure

C; cohesion of soil (kN/m^2)

$\emptyset$; Angle of internal friction

W; Weight of slice (kN)

l ; Length of slice circle (m)

b; Width of slice (m)

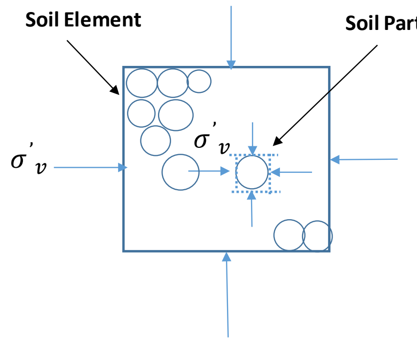
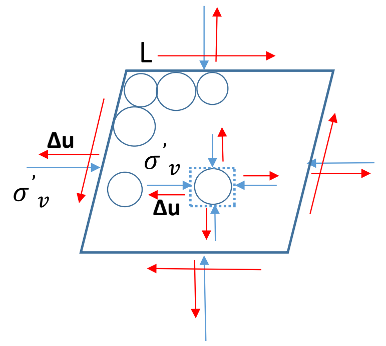
u; Pore water pressure usually generated by ground water (kN /m^2)

Δu; Excessive pore water pressure induced by an earthquake (kN /m^2)

α; The angle of the normal and the vertical lines that can be placed in the center of the arc(°)

Here, C is reduced to 0.0 when particular soil layer is completely liquefied , i.e, FL≦1.0. This is the modification to the original equation stated in “Chapter 6. A River Embankment against an Earthquake” by Japan Institute of Country–ology and engineering. In this original equation, C is assumed to have initial value even when FL ≦1.0.

Fig 2.3.1 shows the outline of liquefaction process.

Before Earthquake	During Earthquake
	
･σ'_v: Effective Stress (confining pressure for soil particle) ($\sigma'_v = \sigma_v - u$) ･σ_v: Total Stress ･u: Pore water pressure usually generated by ground water	･Δu: Excessive pore water pressure generated by Seismic Shear Stress L ･Effective stress σ'_v (confining pressure for soil particle) is lost by Δu. (For width of one slice b for slope stability, effective stress becomes（W－u･b）－（Δu･b）） ･When Δu reach σ'_v , soil particle loses its confining pressure completely ending up complete liquefaction. .

Mechanism of Generation of Excessive Pore Water Pressure Δu during an Earthquake

① When shear stress is induced to soil element during earthquake, the soil element is forced to be under shear deformation.

② In the case of saturated loose sand , when soil element is put under shear deformation , one soil particle of the element tries to settle into the room among other soil partiles.

③ Then, soil element itself tries to decrease its bolume.

④ However, during an earthquake under very short time strong shear force , soil element is put under undrained condition , in other words, decreasing bolume change of the element is completelty constrained.

⑤ This prohibition of bolumetric change of soil element leads to the generation of inner water pressure as the reaction to constraint of bolumetric change.

⑥ This inner water pressure generated as the reaction to constraint of bolume change of the soil element is Excessive Pore Water Pressure Δu.

Fig 2.3.1 Outline of Liquefaction Process

2.3.2 Minimum Safety Factor (Fsd(K_h)) of the Slope Stability of an Embankment against Horizontal Force with Design Seismic Intensity (without liquefaction)

Minimum Safety Factor of the Slope Stability of the Embankment when it is not suffered liquefaction during an earthquake is calculated from following equation.

Here, " not suffering liquefaction during an earthquake " means " No excessive pore water pressure Δu is generated during an earthquake at any soil layer"

$$F_{sd}(K_h) = \frac{\Sigma[c \cdot l + \{(W - u \cdot b) \cdot \cos\alpha - K_h \cdot W \sin\alpha\} \cdot \tan\emptyset]}{\Sigma[W \cdot \sin\alpha + K_h \cdot W \cdot (y/r)]}$$

Where K_h ; The design seismic intensity due to the inertia force

y; The height from center of gravity of slice to center of sliding circle (m)

r; Radius of sliding circle (m)

2.3.3 Subsidence Amount (Maximum) of an Embankment due to an Earthquake

Estimation method of subsidence amount of an embankment during and after an earthquake is provided in Japan based on the accumulated data measured in the past.

Table 2.3.3.1 shows the relation between subsidence amount and minimum safety factor of slope stability.

Table 2.3.3.1 Relation between Subsidence Amount (maximum) and Fsd

$F_{sd}(K_h)$	$F_{sd}(\Delta u)$	Subsidence Amount (maximum)
$1.0 < F_{sd}(K_h)$	$1.0 < F_{sd}(\Delta u)$	0
$0.8 < F_{sd}(K_h) \leqq 1.0$	$0.8 < F_{sd}(\Delta u) \leqq 1.0$	Embankment height × 0.25
$F_{sd}(K_h) \leqq 0.8$	$0.6 < F_{sd}(\Delta u) \leqq 0.8$	Embankment height × 0.50
—	$F_{sd}(\Delta u) < 0.6$	Embankment height ×0.75

3. Case Study on the Embankment at Bogra

3.1 Calculation process of slope stability of the embankment at Bogra against an earthquake

Minimum safety factors of slope stability $F_{sd}(\Delta u)$ and $F_{sd}(K_h)$ can be obtained based on the equations previously described in 2.3.1.2 and 2.3.2. For these equations ,seismic coefficient Ks and excessive pore water pressure Δu are required.

According to the seismicity map of Bangladesh, the seismic intensity at Bogra is specified as 0.15. On the other hand, in Japan, design seismic intensity for a river embankment is specified as described in Table 3.1.

Here, in the Case Studies on the embankment at Bogra , the seismic intensities (Ks) are selected as 0.15 and 0.18.These are medium size site specific seismicity (0.15) at Bogra and strong seismic intensity (0.18) for evaluating allowable earthquake resistance.

Table 3.1 Design Seismic Intensity Specified for an Embankment in Japan

	Embankment Size	Categorized Area of Sesmic Intensity		
		Strong Earthquake Area	Medium Eartquake Area	Small Eartquake Area
Design Seismic Intensity for Liquefction of the Underground		0.18	0.15	0.12
Design Seismic Intensity for Inertial Force for an Embankment	10≧B /H	0.18	0.15	0.12
	20≧B/H＞10	0.16	0.14	0.11
	B/H＞20	0.15	0.12	0.1

B: Width of Embankment Base, H: Height of Embankment

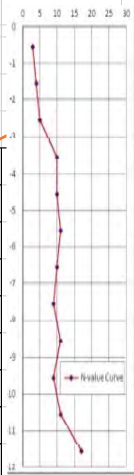
On the other hand, Δu can be calculated in accordance with the process described from 2.2.1 through 2.3.1.1.

Table 3.2 shows basic information of underground soil condition of the embankment site at Bogra and excessive pore water pressure Δu at each soil layer of the ground induced by an earthquakes with seismic intensities Ks of 0.15 and 0.18.

As for actual calculation of the minimum safety factors of slope stability, computer software for slope stability were adopted.

Fig3.1 and Fig 3.2 show the minimum safety factors of slope stability of the embankment at Bogra during the earthquakes with seismic intensity Ks of 0.15 and 0.18 respectively.

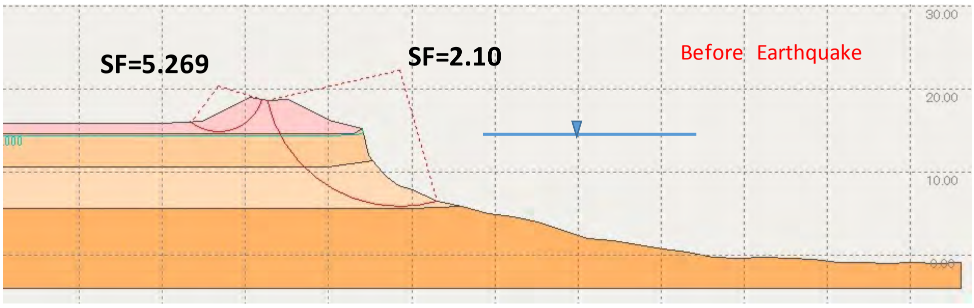
Table 3.2 Basic Underground Soil Condition at Bogra and induced Excessive Pore Water Pressure during Earthquakes



Basic information of underground soil condition of the Embankment at Bogra													Ks=0.15 (Design Seismic Intensity)		Ks=0.18 (Design seismic Intensity)	
Level (m) (PWD)		X0 (m)	X(m)	hw(m)	FC(%)	N	ρ_{t1} (kN/m³)	ρ_{t2} (kN/m³)	σ_v (kN/m²)	σ'_v (kN/m²)	c (kN/m²)	ϕ (°)	FL	Δu(kN/m²)	FL	Δu(kN/m²)
		Depth from Top of the Embankment	Depth from Ground Surface	Underground Water Level from Ground Surface	Fine particle Contents	N value	Density of soil in shallower than water tble position	Density of soil in deeper than the water table position	Total stress	Effective stress	Cohesion	Inner friction angle	Resistivity against Liquefaction	Excessive Pore Water Pressure induced by an earthquake	Resistivity against Liquefaction	Excessive Pore Water Pressure induced by an earthquake
18.94	Top of Embankment	0					Top of Embankment									
		- 1														
	Ground Surface	- 2			Ground Surface											
15.59		- 3	0													
14.339		- 4	- 1	- 1.251	75	10	17.934	19.11	17.64	20.1	23.88	33.13	N/A(FC>35%)	0	N/A (FC>35%)	0
	Underground Water Level (after the strong flood period)	- 5	- 2		30	11	17.934	19.11	36.75	29.41	23.88	33.13	2.944	0.0153	2.453	0.055
		- 6	- 3		30	10	17.934	19.11	55.86	38.72	23.88	33.13	1.705	0.923	1.421	3.307
		- 7	- 4		45	9	17.934	19.11	74.97	48.03	23.88	33.13	N/A(FC>35%)	0	N/A (FC>35%)	0
		- 8	- 5		45	10	17.934	19.11	94.08	57.34	23.88	33.13	N/A(FC>35%)	0	N/A (FC>35%)	0
		- 9	- 6		13	9	17.934	19.11	113.19	66.65	23.88	33.13	1.007	63.3	0.84	66.65
		- 10	- 7		13	11	17.934	19.11	132.3	75.96	23.88	33.13	1.067	48.22	0.889	75.96
		- 11	- 8		4	15	17.934	19.11	151.41	85.27	23.88	33.13	1.163	29.57	0.969	85.27
		- 12	- 9		4	17	17.934	19.11	170.52	94.58	23.88	33.13	1.206	25.44	1.005	91.17
		- 13	- 10		4	17	17.934	19.11	189.63	103.89	23.88	33.13	1.178	32.97	0.982	103.89

Before Earthquake

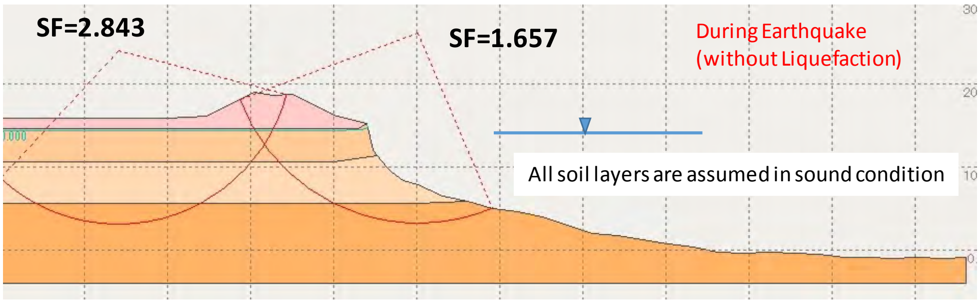
water level = 4.339m



During Earthquake
(Ks=0.15)

Without Liquefaction
SF = $F_{sd}(K_h)$

water level = 4.339m



During Earthquake
(Ks=0.15)

With Liquefaction
SF = $F_{sd}(\Delta u)$

water level = 4.339m

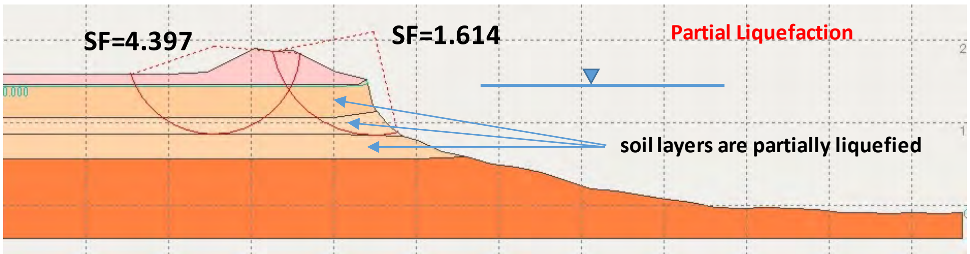
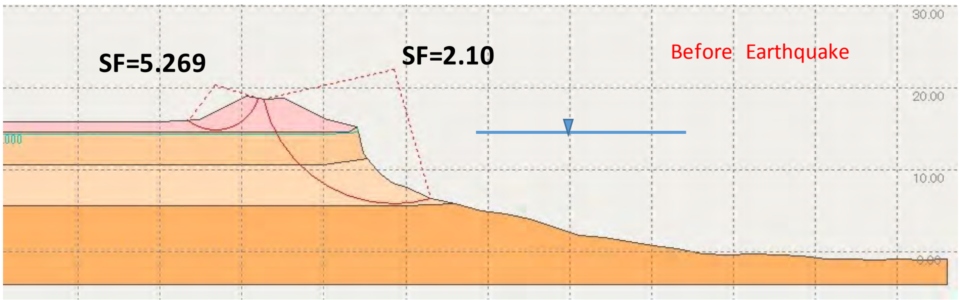
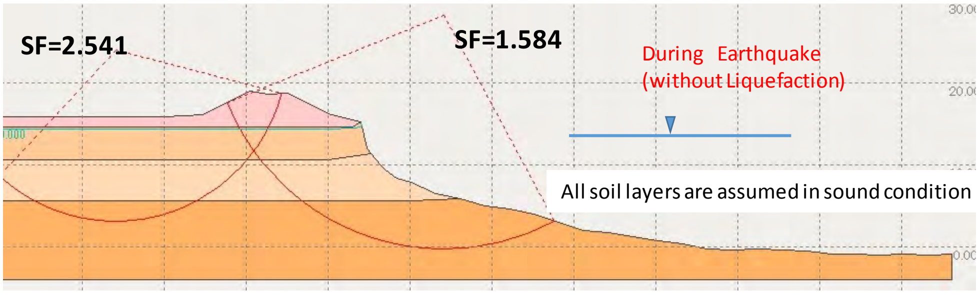


Fig 3.1 Safety Factor of Slope Stability of the Embankment at Bogra (Ks=0.15)

<table>
<tr>
<td>Before Earthquake

water level = 4.339m</td>
<td>
</td>
</tr>
<tr>
<td>During Earthquake
(Ks=0.18)

Without Liquefaction
SF = $F_{sd}(K_h)$

water level = 4.339m</td>
<td>
</td>
</tr>
<tr>
<td>During Earthquake
(Ks=0.18)

With Liquefaction
SF = $F_{sd}(\Delta u)$

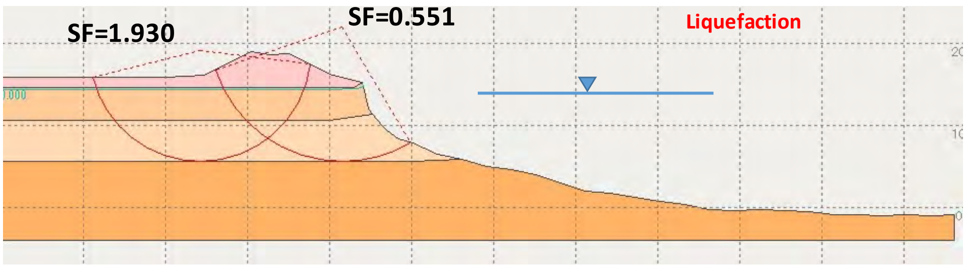
water level = 4.339m</td>
<td>
</td>
</tr>
</table>

Fig 3.2 Safety Factor of Slope Stability of the Embankment at Bogra (Ks=0.18)

3.2 Slope Stability and associated Subsidence of the embankment at Bogra against an earthquake

3.1.1 Case Study of Ks=0.15 (Site specific seismic intensity at Bogra)

- Slope stability of the embankment is assessed to be secured against the site specific seismic intensity at Bogra (Ks=0.15). Minimum safety factor Fsd(K_h) when liquefaction is not considered is 1.657. And minimum safety factor Fsd(Δu) when liquefaction is considered is 1.614.
- When the subsidence during and after the earthquakes is assessed based on the estimation method available in Japan, it is found that the subsidence is unlikely to occur.

3.1.2 Case Study of Ks=0.18 (Seismic intensity equivalent to strong earthquake specified for an river embankment in Japan)

- Slope stability of the embankment is assessed to be not secured, unstable, if it is subject to strong earthquake specified in Japan. Minimum safety factor Fsd ($K_{h)}$ when liquefaction is not considered is 1.584. However, minimum safety factor Fsd (Δ u) when liquefaction is considered is 0.551.
- When the subsidence during and after the earthquake is assessed based on the estimation method , it is found that subsidence will end up 2.513m , 75.0% of embankment height (3.35 x 0.75).

4. Conclusion

In this report, assessment method of the earthquake resistance of an embankment was developed and case studies on the embankment at Bogra were performed.

From these studies, followings were clarified.

(1) Assessment method on slope stability against an earthquake

- Assessment method of earthquake resistance of the embankment available in Japan can be applied to the embankments in Bangladesh using basic information on soil characteristics and seismic intensity map available in Bangladesh.

(2) Slope Stability and associated Subsidence of the embankment at Bogra

Case Study of Ks=0.15 (Site specific seismic intensity at Bogra)

- Slope stability of the embankment is assessed to be secured against the site specific seismic intensity at Bogra (Ks=0.15). Minimum safety factor Fsd(K_h) when liquefaction is not considered is 1.657. And minimum safety factor Fsd(Δu) when liquefaction is considered is 1.614.
- When the subsidence during and after the earthquakes is assessed based on the estimation method available in Japan, it is found that the subsidence is unlikely to occur.

Case Study of Ks=0.18 (Seismic intensity equivalent to strong earthquake specified for an river embankment in Japan)

- Slope stability of the embankment is assessed to be not secured, unstable, if it is subject to strong earthquake specified in Japan. Minimum safety factor Fsd (K_h) when liquefaction is not considered is 1.584. However, minimum safety factor Fsd (Δu) when liquefaction is considered is 0.551.
- When the subsidence during and after the earthquake is assessed based on the estimation method , it is found that subsidence will end up 2.513m , 75.0% of embankment height (3.35 x 0.75).

Bangladesh Water Development Agency
The People's Republic of Bangladesh

THE PROJECT FOR CAPACITY DEVELOPMENT OF MANAGEMENT FOR SUSTAINABLE WATER RELATED INFRASTRUCTURE IN THE PEOPLE'S REPUBLIC OF BANGLADESH

ASSESSMENT ON STABILITY OF EXISTING EMBANKMENT WHICH IS SUBJECT TO FLOOD WATER

July 2015

JICA EXPERT TEAM

CONTENTS

1. Objective

The purposes of this report are, first of all, assessing slope stability and seepage safety of existing embankments of Bangladesh which are subject to strong flood water, and secondary, identifying triggering factor of damages of embankments in the past.

According to the site surveys of existing embankments, damages of the embankments during strong flood period seem to be mainly triggered by prior incremental erosion from the edge of setback in front of embankment which had been subject to river current during entire rainy season.

In other words, in almost all embankments, prior incremental erosion by river currents during entire rainy season dominates the fatal damages of embankments during subsequent strong flood period.

Therefore, slope stability and seepage safety of the embankment during strong flood period is not clarified if the embankment is protected against prior incremental erosion due to river current during entire rainy season.

In this report, first of all, slope stability and seepage safety of existing embankments which are subject to strong flood period are assessed by seepage flow analysis and slope stability analysis under the condition that the embankment is protected against prior incremental erosion due to river currents during entire rainy season.

For these analyses, soil investigations at existing embankments, topological surveys, observed water levels and rain falls during strong flood period were used.

Secondary, as an additional study, impact of incremental erosion by river current on the damage of embankments in the past is studied by seepage flow analysis and slope stability analysis using the simulation model in which part of setback in front of sound embankment is pre eliminated for modeling incremental erosion.

Prior to the site survey, FS (Feasibility Study) had been conducted to investigate characteristics of the embankments at twelve (12) locations in Bangladesh. In the FS, soil characteristics around top of the embankments had been investigated. Fig1.1 shows locations for the FS and Table1.1 shows soil characteristics around top of the embankments of twelve (12) locations evaluated by the FS.

Following the FS , JICA expert team conducted this site survey and finally concluded that embankments of Bangladesh cannot be discussed nor uniformly specified in terms of their hydrological aspects, soil characteristics aspects, river condition aspects.

Therefore, JICA expert team finally categorized entire embankments of Bangladesh into following four (4) types.

Type A (Embankments along major rivers)
Type B (Embankments in high tidal waves)
Type C (Embankments in Haor area)
Type D (Embankments affected by flash flood)

Fig 1.2 and Table 1.2 show distribution of embankments and soil characteristics around top of embankments finally categorized by this site survey.

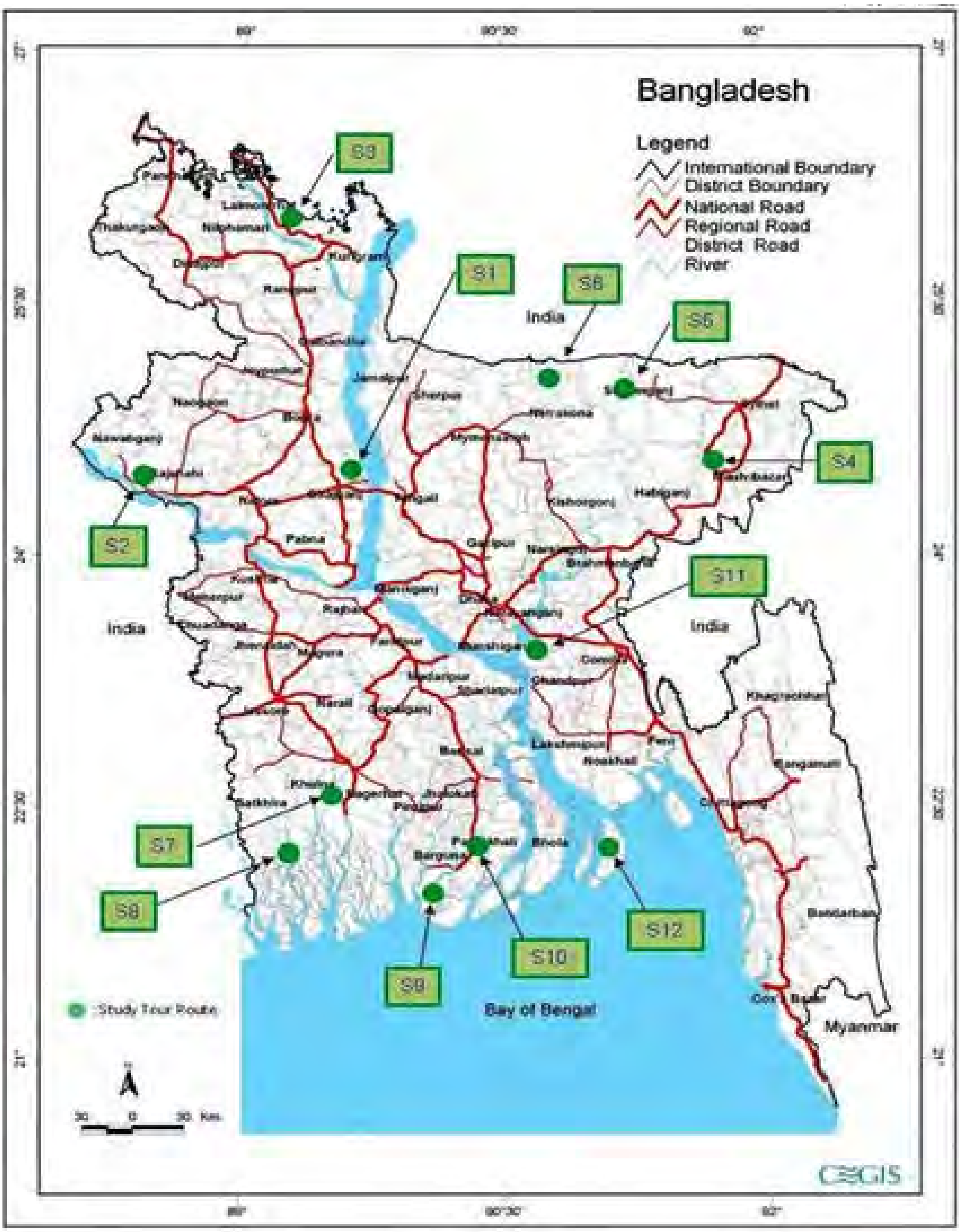


Fig1.1 Location of Investigation Site in FS

Table1.1 Soil Characteristics at Locations in FS (Grain Size Distribution)

Site No.	Location	Sand(%)	Silt(%)	Clay(%)
S-1	Sirajganj	10	85	5
S-2	Rajshahi	0	96	4
S-3	Rangpur	62	38	0
S-4	Moulvibazar	0	70	30
S-5	Sunamgong	0	60	40
S-6	Netrokona	0	50.5	49.5
S-7	Khulna	0	72.5	27.5
S-8	Sathira	0	71.5	28.5
S-9	Bargna	0	79.5	20.5
S-10	Patuakhali	0	86.5	13.5
S-11	Chandpur	40	57	3
S-12	Noakhali	0	83.5	16.5

《 Categories of Embankment 》

A-type	Embankments along 3major Rivers
B-type	Embankments in Tidal area
C-type	Embankment in Haor area
D-type	Embankment in Flash flood area (Eastern / Northern parts of country)

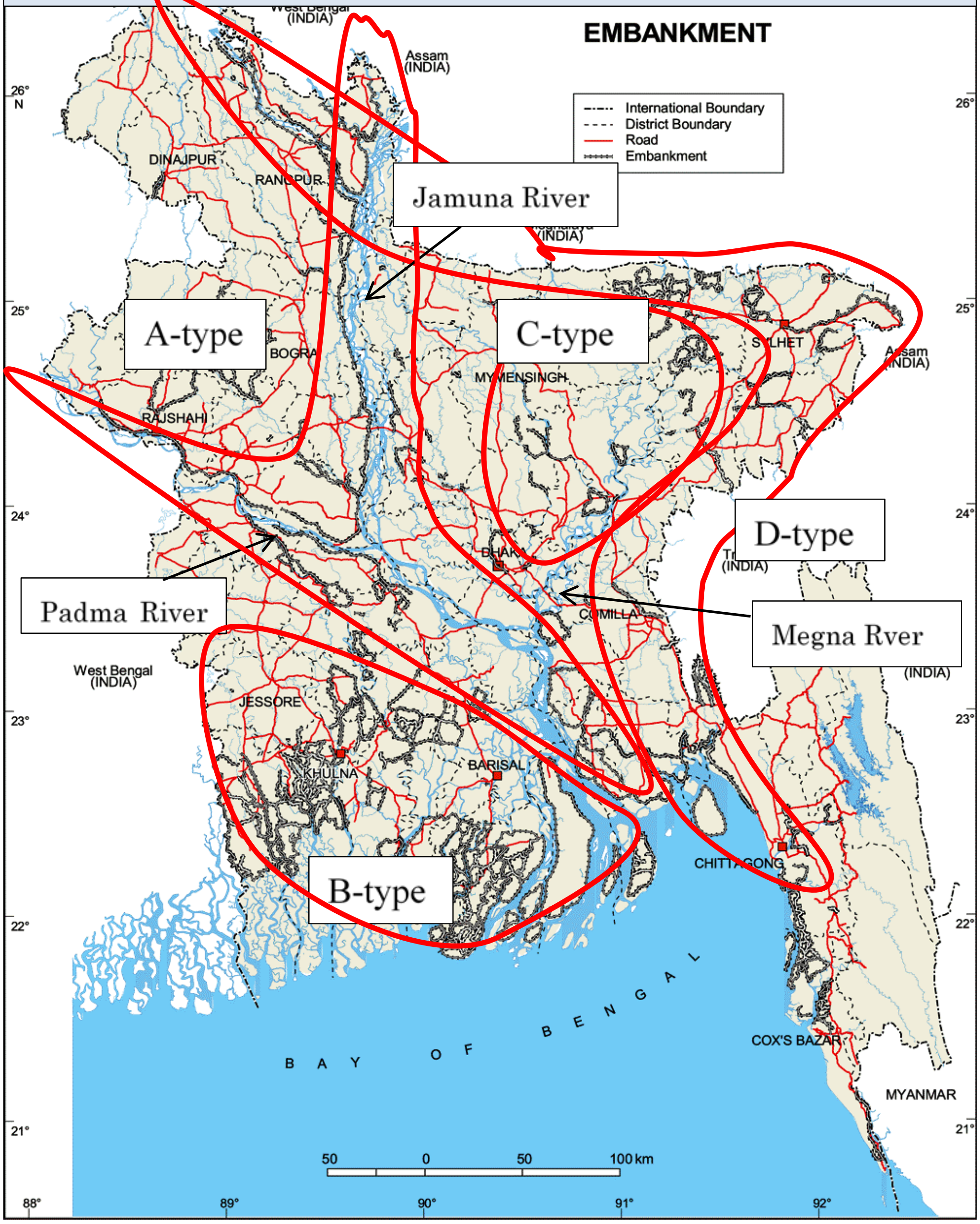


Fig 1.2 Categorized Embankments in Bangladesh

Table 1.2 Soil Characteristics of Categorized Embankments (Grain Distribution)

Type of Embankment	Site No.	Location	Sand(%)	Silt(%)	Clay(%)
A	S-1	Sirajganj	10	85	5
	S-2	Rajshahi	0	96	4
	S-3	Rangpur	62	38	0
	S-11	Chandpur	40	57	3
B	S-7	Khulna	0	72.5	27.5
	S-8	Sathira	0	71.5	28.5
	S-9	Bargna	0	79.5	20.5
	S-10	Patuakhali	0	86.5	13.5
	S-12	Noakhali	0	83.5	16.5
C	S-5	Sunamgong	0	60	40
	S-6	Netrokona	0	50.5	49.5
D	S-4	Moulvibazar	0	70	30

A:Embankments along major rivers, B:Embankments in high tidal area, C:Embankments in Haor area, D:Embankments affected by flash flood.

2. Seepage Flow Analysis and Slope Stability Analysis

2.1 Embankments for Analysis

For seepage flow analysis and slope stability analysis, typical four (4) embankments were selected from fourteen (14) locations where this site survey were conducted following FS.

In numerical analyses for assessing slope stability and seepage safety of existing embankments, three (3) types A,B,D among above four (4) types were simulated and assessed for strong flood period.

The reason C type is eliminated from this numerical analysis is as follows.
According to FS(Feasibility Study) conducted prior to the survey, the embankments for type C are dominantly clay in terms of its soil condition.

With this condition, it was concluded that slope in-stability and seepage un-safety are unlikely to occur in C type. Additionally, it was concluded that the damages of embankments at the area are almost generated by over flow during pre-monsoon period and sometimes by public cut.

Here, Type A is the embankments along major rivers. Typical Type A is that at Chandpur. Type A also includes retired embankment namely Bogra where sand dredged from river bed is used for embankment. This type of embankment is dominantly sand, therefore, slope stability and seepage safety should be carefully studied. Therefore, the embankment at Bogra was included in the embankments for analysis.

These four (4) embankments are two (2) for Type A, one (1) for Type B and one (1) for Type D. Soil investigation on embankment and associated underground were conducted for these four (4) embankments.

Embankments used for assessment on slope stability and seepage safety are listed below.

1. Type A-1 Bogra (Latitude 24° 47′ 48″, Longitude 89° 36′ 11″)
2. Type A-2 Chandpur (Latitude 23° 11′ 29″, Longitude 90° 38′ 35″)
3. Type B Khulna (Latitude 22° 37′ 35″, Longitude 89° 27′ 47″)
4. Type D Moulvibazar (Latitude 24° 32′ 11″, Longitude 91° 44′ 09″)

2.2 Methods for Seepage Flow Analysis and Slope Stability Analysis

Underground water seepage analysis (unsteady seepage flow analysis) and associated slope stability analysis were conducted for the assessment.

Computer software named [SAUSE Version 3.1, commercial product available in Japan by NITA CONSULTANT] was applied to numerical analysis. This software was developed in accordance with “Chapter 4.Structural Integrity of River Embankment” by Japan Institute of Country –ology and engineering.

In this software, slope stability analysis is conducted at each different time with associated time dependent seepage line generated by unsteady seepage flow analysis. Finite Element Method (FEM) is employed for this software.

Outline of methods for seepage flow analysis and slope stability analysis employed in this software are described as follows.

(1) Unsteady Seepage Flow Analysis

Fig 2.1 shows dominant equation for unsteady seepage flow obtained from Law of Conservation of Mass, and Darcy’s Law.

Typical characteristics of the dominant equation is existence of time dependent term on the right hand side of the equation. This term expresses time dependent variation of Sr (degree of saturation), n (porocity of soil). With this equation, rain fall change and water level change can be reasonably expressed.

As for the input of water level and rain fall during strong flood period for numerical simulation on unsteady seepage flow analysis and its associated slope stability analysis, actual time dependent elapsed data were applied to figure out real status of embankments during strong flood period.

In case of steady seepage flow analysis, the term at the right hand side will be eliminated.

(2) Slope Stability Analysis

Fig 2.2 shows outline of evaluation method of minimum safety factor in slope stability analysis. Minimum safety factor of slope stability is evaluated at each different time in accordance with changing seepage line generated from unsteady seepage flow analysis. Here, minimum safety factor at each particular time is identified by automatically searching the circular sliding with minimum safety factor.

Equation of Continuity (Law of Conservation of Mass)

$$\frac{\partial(Sr \cdot n \cdot \rho w)}{\partial t} + \frac{\partial(\rho w \cdot Vx)}{\partial x} + \frac{\partial(\rho w \cdot Vy)}{\partial y} + \frac{\partial(\rho w \cdot Vz)}{\partial z} = 0$$

here

Sr: degree of saturation , **n**: porocity, ***ρ*w**: unit density of pore water

Vx,Vy,Vz: fluid velocity of pore water

Equation of Motion (Darcy's Law)

here

$Vx=-Kx\frac{\partial H}{\partial x}$ 、$Vy=-Ky\frac{\partial H}{\partial y}$ 、 $Vz=-Kz\frac{\partial H}{\partial z}$

Kx,Ky,Kz: coefficient of permeability H: total head

From these above

Dominant equation for Unsteady Seepage Flow

$$\frac{\partial}{\partial x}\left(Kx\frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(Ky\frac{\partial H}{\partial y}\right) = \frac{\partial}{\partial t}(Sr \cdot n)$$

Fig 2.1 Dominant Equation for Unsteady Seepage Flow

Safety factor for Slope Stability

$$Fs=\Sigma\{c\,l+(w-ub)\cdot\cos\alpha\cdot\tan\phi\}/(w\cdot\sin\alpha)$$

here

u: pore water pressure at circular sliding line

w: mass of divided cross section of embankment

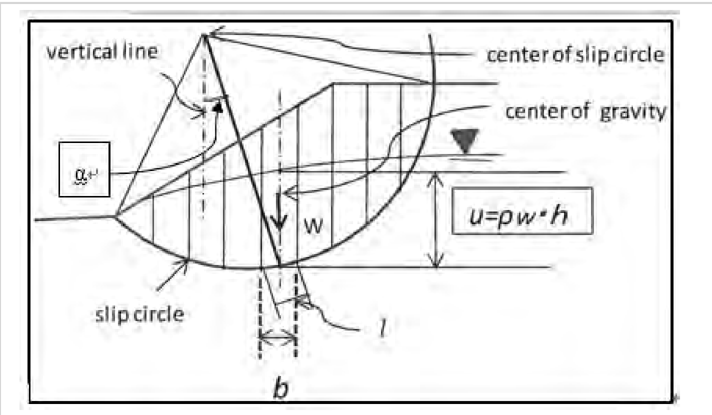
c: cohesion of soil along circular sliding line

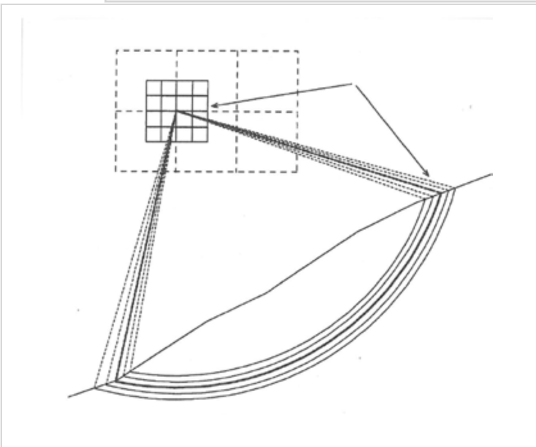
l: length of circular sliding line

ϕ: inner friction angle of soil

b: width of divided cross section of embankment

α: angle between perpendicular and vertical lines to circular sliding





Automatically Searched Circular Sliding with Minimum Safety Factor

Fig 2.2 Outline of Slope Stability Analysis

2.3 Conditions for Seepage Flow Analysis and Slope Stability Analysis

2.3.1 Parameters for Analysis on Soil Characteristics

Based on soil investigation and laboratory tests, parameters for numerical analysis were specified as described on Table 2.3.1.1 and Table 2.3.1.2. Fig 2.3.1.1 shows N values at the Embankments.

In these tables, C (cohesion) , ϕ(inner friction angle) and coefficient of permeability were decided in accordance with following policy.

(1) C,ϕ

In case permeability is low

In case that soil constituency is dominantly clayey silt and therefore permeability is very low, un-drained shear strength Cu,ϕu should be applied. Here, Cu=Ccu + σtanϕcu (Ccu and ϕcu are cohesion and shear strength increasing rate with the depth identified from enveloping line of Mohr's Circles with total stress expression. These Mohr's Circles are obtained from Consolidated Un-drained Tri-axial Compression Tests (CU tests). ϕu is assumed to be zero (0.0).

These Cu and ϕu are applied to embankment part and underground part of Moulvibazar, Khulna and embankment part of Chandpur.

In case permeability is high

In case that soil constituency is dominantly sand and therefore permeability is high, drained shear strength C and ϕ should be applied.

Drained shear strength C and ϕ should be obtained from CU test or direct shear test using undisturbed samples. However, it is almost impossible to obtain undisturbed sample for sandy soil, therefore, on sandy soil, direct shear tests were conducted using disturbed samples. The test results C,ϕ from direct shear tests with disturbed sample tend to be low compared to those by undisturbed samples. However, the specific ratio between those from undisturbed samples and those from disturbed samples cannot be specified. Therefore, for numerical analyses this time, the results of direct shear tests may be applied for numerical analyses from the stand point that the results may provide safety side results in terms of slope stability and seepage safety.

These C and ϕ are applied to embankments part and underground part of Bogra , und underground part of Chandpur.

(2) Coefficient of Permeability

Basically , coefficient of permeability obtained from laboratory tests with undisturbed samples should be applied. In case there is no laboratory test results due to the lack of undisturbed samples, the coefficients may be specified from Cregar's assumption

Table 2.3.1.1 Parameters of Soil Characteristics for Numerical Analyses (Shear strength and Permeability)

Location	Part of Embankment	Shear Strength				Coefficient of Permeability			
		drained shear strength		undrained shear strength		Laboratory test		Creager's assumption	
		φ (°) average	C(kN/m²) average	φu (°) average	Cu(kN/m²) average	kx(cm/s)	ky(cm/s)	Particle size of 20% passing thieve	Creager's assumption(cm/s)
Bogra	embankment	28.5	27.36					0.01～0.03	4.53E-05
	underground	33.13	23.88					0.03～0.09	6.93E-04
Chandpur	embankment			0	14.33			0.001～0.0045	2.50E-06
	underground	29.65	26.87					0.002～0.08	7.50E-04
Khulna	embankment			0	11.65		9.76E-07	0.003～0.009	
	underground			0	19.64				3.00E-07
Moulvibazar	embankment			0	29.39		5.30E-06	＜0.001	
	underground			0	26.07	6.52E-06	9.25E-05	～0.0017	

Table 2.3.1.2 Parameters of Soil Characteristics for Numerical Analysis (Particle Distribution and Basic Characteristics)

Location	Part of Embankment	Particle distribution (%) Average			Water Content	Soil particle Unit Density	Wet Unit Density	Dry Unit Densit y	Void Ratio	Saturated Unit Density
		sand (0.0075 mm～)	silt (0.002～0.0075mm)	clay (～0.002 mm)	W (%)	Gs (gf/cm³)	γt (gf/cm³)	γd (gf/cm³)	e	γsat (gf/cm³)
Bogra	embankment	52	43	5	16.9	2.67	1.79	1.53	0.75	1.96
	underground	77	20	3	19.4	2.6	1.83	1.54	0.69	1.95
Chandpur	embankment	8	73	19	22.1	2.65	1.88	1.54	0.72	1.96
	underground	66	29	5	26.5	2.65	1.93	1.52	0.74	1.95
Khulna	embankment	6	64	30	33	2.48	1.92	1.45	0.71	1.87
	underground	16	53	31	41.4	2.51	2.08	1.49	0.68	1.9
Moulvibazar	embankment	30	51	19	20.1	2.68	1.97	1.63	0.64	2.02
	underground	7	64	29	43.1	2.6	1.74	1.26	1.06	1.78

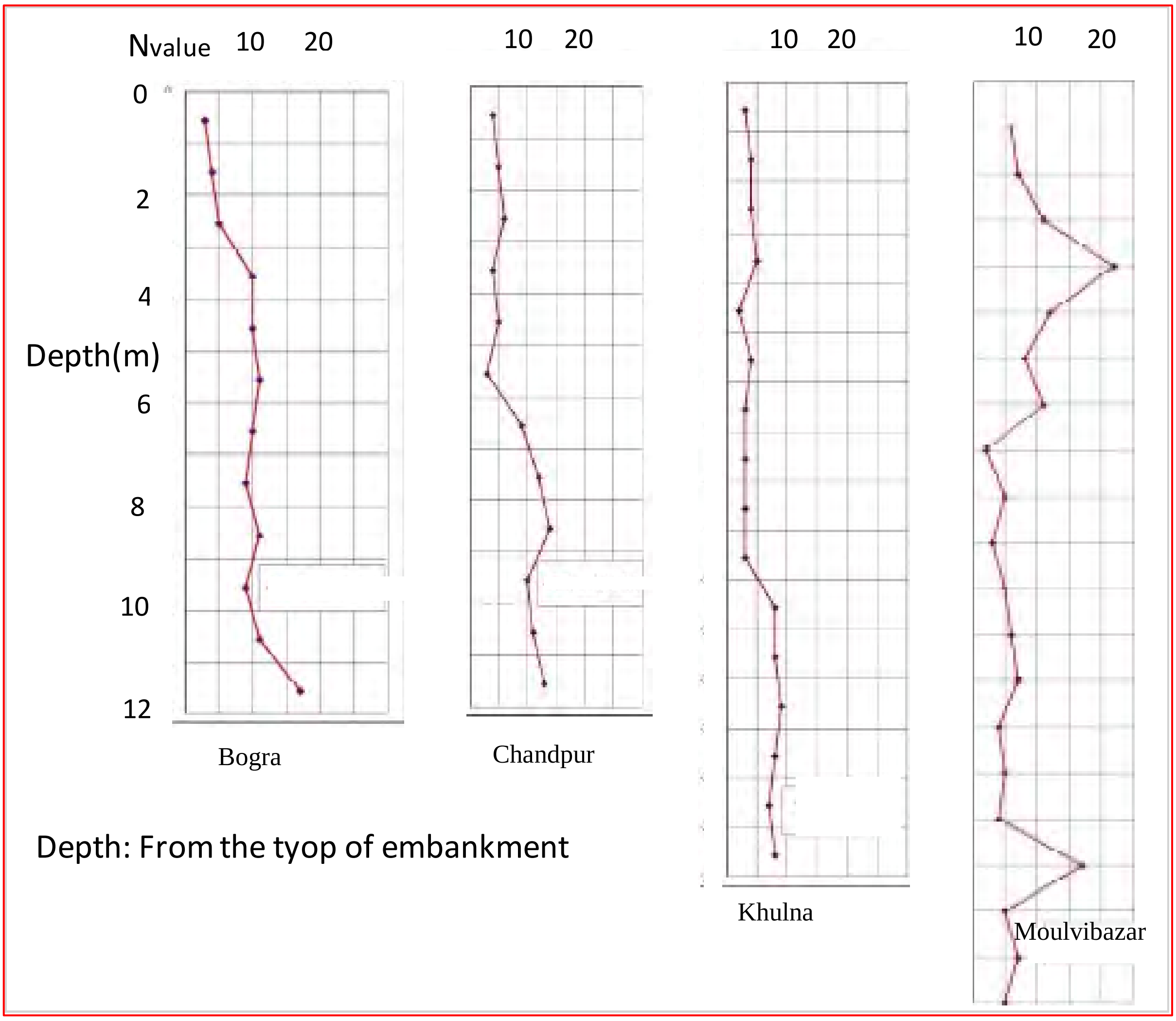


Fig 2.3.1.1 N values at Embankments

2.3.2 Water Level and Rain Fall Intensity during Strong Flood Period

(1) Water Level for Numerical Analysis

Water Level Hydrograph (water level change in accordance with elapsed time) applied for analysis at each embankment was selected from accumulated observed water level data obtained at nearby observation points. This **Water Level Hydrograph** was selected from the standpoint that the highest water level of **Water Level Hydrograph** during strong flood period shows maximum value during past around 20 years.

This **selected Water Level Hydrograph** was corrected to that at the embankment location considering the difference of distance and height between the embankment location and observation station.

This **corrected Water Level Hydrograph** was further adjusted so that the highest water level of the corrected selected Water Level Hydrograph ends up **Design Highest Water Level** during strong flood at the embankment.

Here**, Design Highest Water Level** was specified as the level which is 1.0m lower than the embankment height. Here, 1.0m is allowable height.

When highest level of **further adjusted Water Level Hydrograph** is higher than embankment height minus (-) allowable height (1.0m), the further adjusted **Water Level Hydrograph** was used, but should less than embankment height.

When highest level of **further adjusted Water Level Hydrograph** is lower than embankment height minus (-) allowable height, the **further adjusted Water level Hydrograph** was again **finally adjusted** to embankment height minus (-) allowable height.

(2)Rain fall intensities for numerical Analysis

For time dependent rain fall intensities, observed rain fall intensities obtained along with adopted observed water level data were used.

Table 2.3.2.1 shows design water levels of Bogra, Chandpur, Khulna,and Moulvibazar specified based on the observed water levels during strong flood period.

Figures from 2.3.2.1 to 2.3.2.4 show water levels and rain fall intensities specified for the numerical simulations. These values are changing in accordance with elapsed time during strong flood period.

Table 2.3.2.1 Specified Water Levels for Numerical Simulations

Location	Observation Points	Year	Month	Day	Observed highest water level (m)	Embankment hight	specified highest water level for numerical simulation(m)
Bogra	Bahadrabad St. Sirajgang49 St	1988	8	30	17.76	18.94	17.94
Chandpur	Chandpur St Davlatkhan St	1998	9	9	5.62	6.81	5.81
Khulna	Chalna St.	2001	9	18	4.22	3.34	3.34
Moulvibazar	Moulvibazar St	1993	6	8	11.69	12.825	11.825

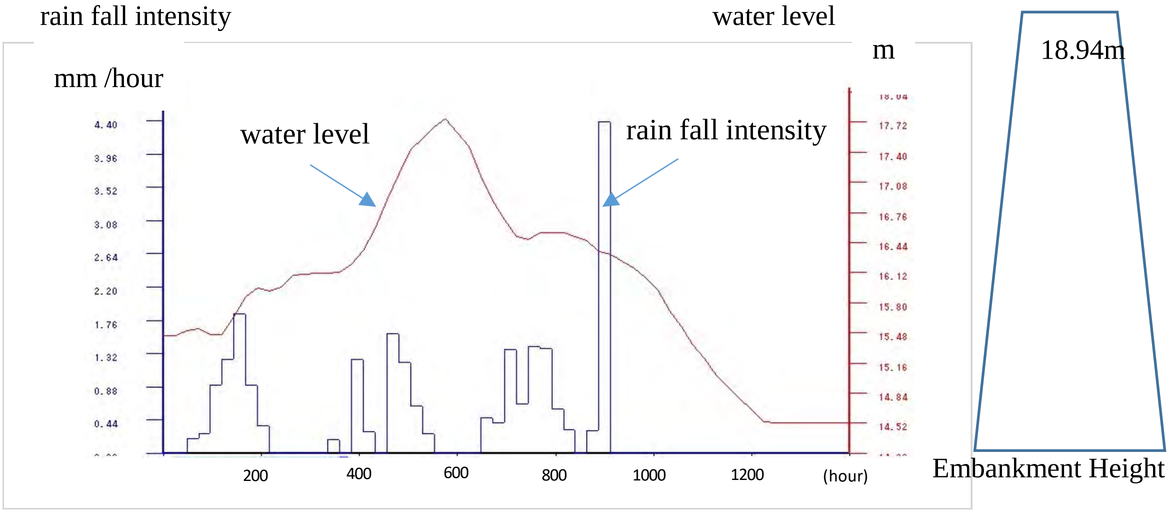


Fig 2.3.2.1 Water levels and rain fall intensities specified for the numerical simulations (Bogra)

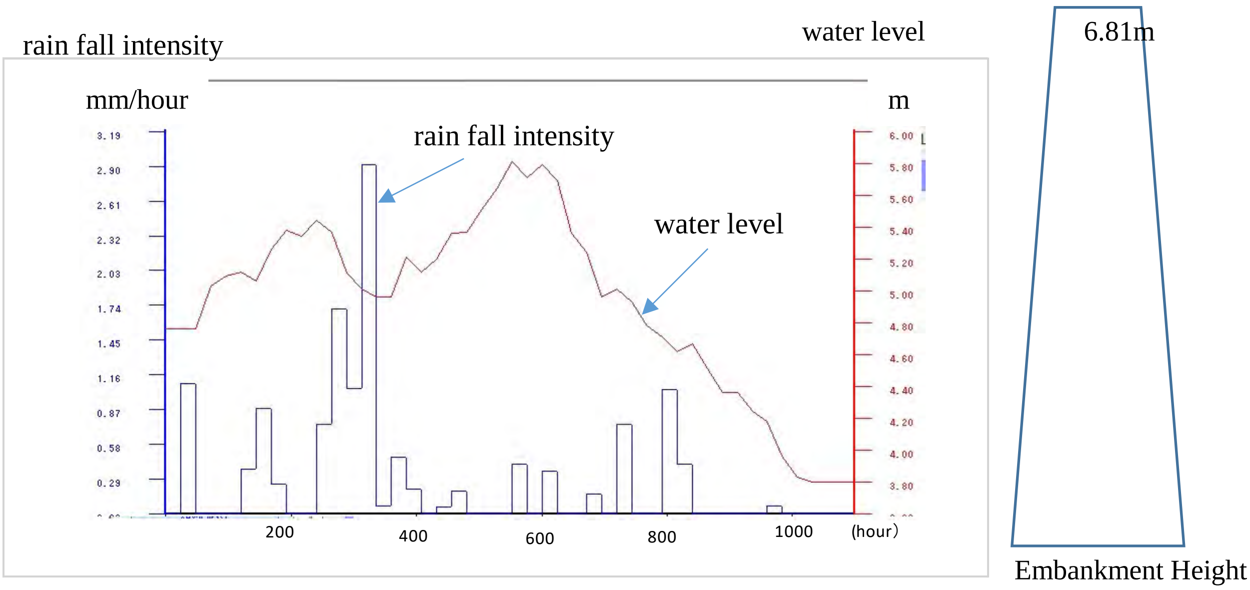


Fig2.3.2.2Water levels and rain fall intensities specified for the numerical simulations (Chandpur)

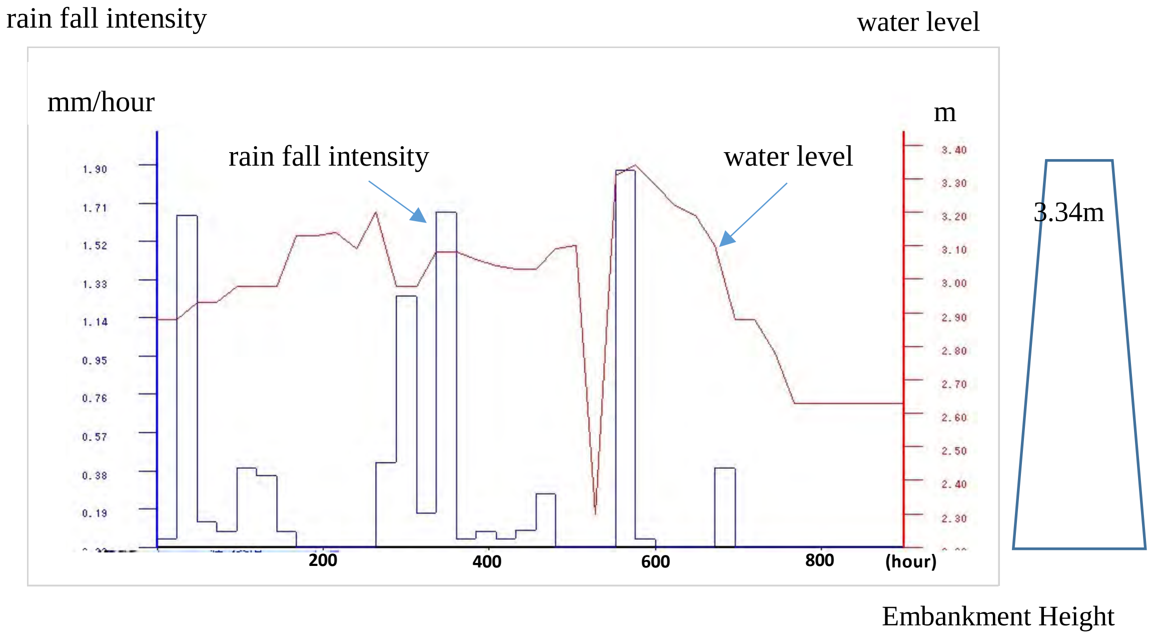


Fig2.3.2.3 Water levels and rain fall intensities specified for the numerical simulations (Khulna)

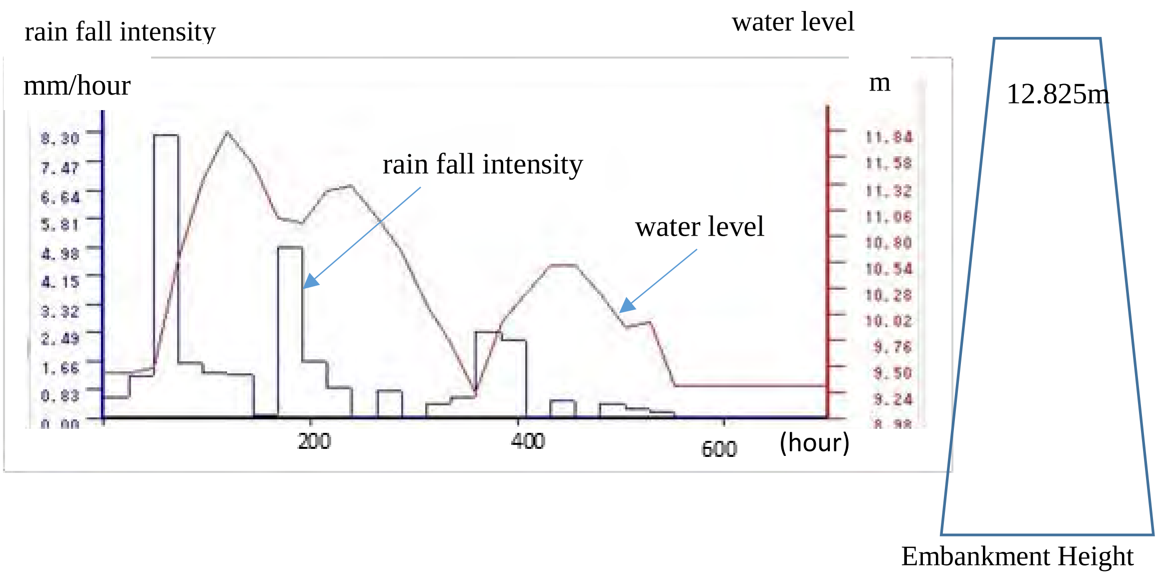


Fig2.3.2.4 Water levels and rain fall intensities specified for the numerical simulations (Moulvibazar)

2.3.3 Methods for Assessment on Slope Stability and Seepage Safety

(1) Method for Assessment on Slope Stability

Slope Stability was assessed by minimum safety factor of circular sliding of embankment during strong flood period. When minimum safety factor is over 1.5, the slope stability is judged to be secured

(2) Method for assessment on Seepage Safety

Seepage Safety can be assessed by comparing local hydraulic gradient **i (ix,iy)**with critical local gradient **ic** at the toe of back slope on country side . When **i(ix,iy)** is less than **ic** , seepage safety is judged to be secured.

Critical hydraulic gradient **ic** at the ground surface at the toe of embankment is obtained by following equation.

ic= (Gs-1.0) / (1.0 + e)   Gs : Soil particle unit density, e: Void ratio

Seepage safety during strong flood period can be assessed by comparing local hydraulic gradient (**ix** and **iy**) with critical local hydraulic gradient **ic**. When **ix** and **iy** is less than **ic** , seepage safety is judged to be secured.

In this assessment, in addition to this method, seepage safety was assessed by comparing local hydraulic gradient **(ix** and **iy**) with **0.5**. This value **0.5** is specified in "chapter 4. Structural Integrity of River Embankment " by Japan Institute of Country – ology and engineering for assessing seepage safety from the viewpoint that **ic** is around **1.0** and therefore using **0.5** as critical value instead of **ic** may provide conservative assessment result in terms of seepage safety.

When soil is dominantly clayey silt (Khulna and Moulvibazar), piping is unlikely triggered by seepage damage. However, in this assessment, this method for assessment was applied to all target embankments.

3. Results of Analysis

Table 3.1.1, Table 3.2.1, Table 3.3.1, Table 3.4.1 show potential circular sliding and associated seepage line at specific times during strong flood period focusing on starting point of strong flood period , on around the time of highest water level, on around time of strongest rain fall intensity, and on ending of strong flood period

Along with these, minimum safety factor of slope stability, critical hydraulic gradient ,local hydraulic gradient at the toe of back slope on country side are also described in these tables.

3.1 Bogra (Embankment : Sandy Soil, Underground: Sandy Soil)

From Table 3.1.1, followings are clarified.

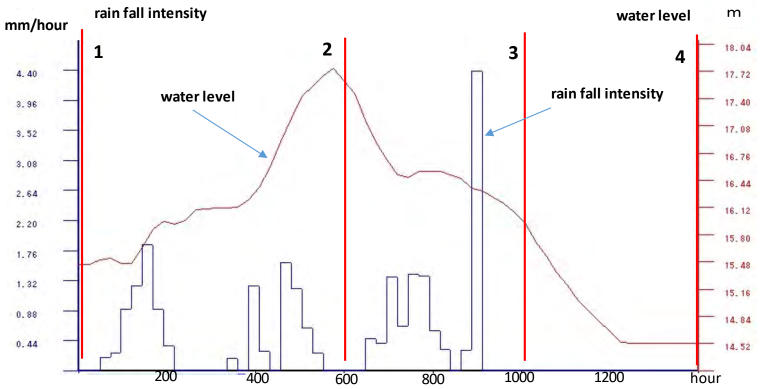
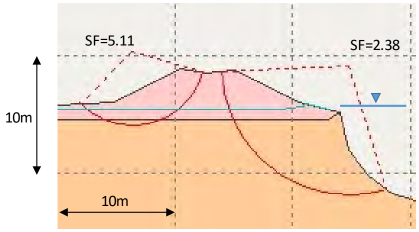
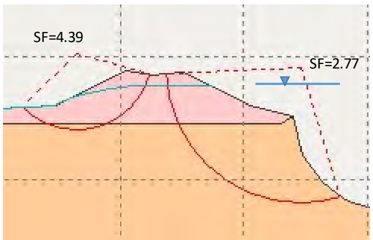
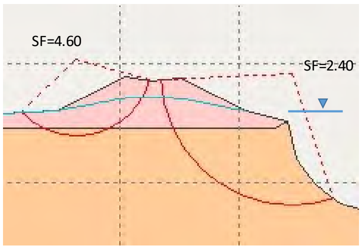
Slope Stability

(1) There are two potential circular slidings, that at back slope on country side of embankment and that at front slope on river side. Among these two, the circular sliding with minimum safety factor is that at front slope on river side.
This seems due to the fact that the edge of setback is near close to the river side having deep water depth.
Safety factor shows over 2.0. Therefore, slope stability is assessed to be secured.

Seepage Safety

(1) Seepage line moves closer toward toe of embankment, emerging on the surface at near the toe of embankment at around time when water level shows highest.
(2) Local hydraulic gradients at the toe of back slope are less than critical hydraulic gradient.
However, the local hydraulic gradient is over 0.5 when water level reaches maximum height. Therefore, embankment at Bogra cannot be assessed being completely free from piping.

Table 3.1.1 Assessment on Slope Stability and Seepage Safety (Bogra)

Water level Rain fall intensity				
	Potential Circular Sliding and Seepage Line	SF(Safety Factor of Slope Stability)		Local Hydraulic Gradient (Back Slope on Country Side)
		Back Slope	Front Slope	Toe of Back Slope
1 (0hour)		5.108	2.377	Horizontal 0 Vertical 0
2 (600hours)		4.388	2.773	Horizontal 0.492 Vertical 0.510
3 (1000hours)		4.595	2.404	Horizontal 0.137 Vertical 0.085
4 (1400hours)		5.262	2.214	Horizontal 0.155 Vertical 0.005

Critical Hydraulic Gradient ic=(Gs−1.0)/(1.0+e) (Bogra :0.95)

3.2 Chandpur (Embankment：Clayey Silt, Underground：Sandy Soil)

From Table 3.2.1, followings are clarified.

Slope Stability

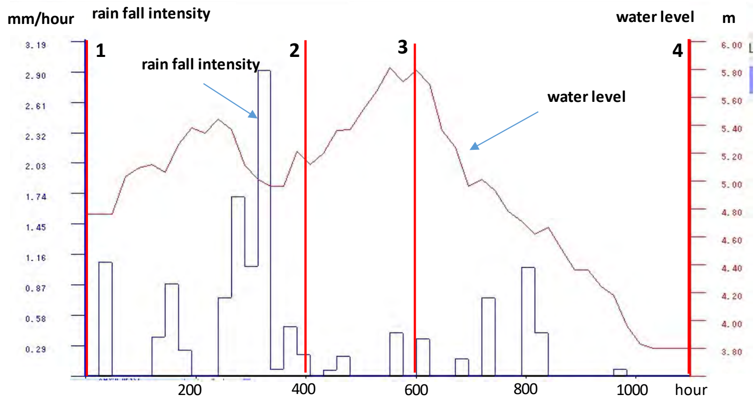
(1) There are two potential circular slidings, that at back slope on country side of embankment and that at front slope on river side. Among these two, the circular sliding with minimum safety factor is that at back slope on country side. This feature is different from Bogra.
This seems due to the fact that embankment at Chandpur has enough length of setback. It may lead to making safety factor of slope stability dominated by seepage flow from front slope on river side toward back slope be minimum.
In this situation, the safety factor is around 1.5. Therefore, slope stability is assessed to be secured.

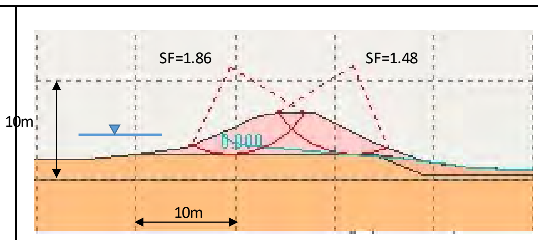
Seepage Safety

(1) Seepage line moves closer toward toe of embankment, emerging on the surface at near the toe of embankment.
(2) When the rain fall intensity is strong, the seepage line appears at the around the surface of top of embankment in addition to that in the embankment (at 400 hours from the start of strong flood period). It seems due to the fact that embankment at Chandpur is dominantly clayey silt with low permeability and strong rain fall does not contribute to making seepage line move upward quickly ending up making rain fall streaming on the surface of top of embankment
(3) Local hydraulic gradient are less than critical hydraulic gradient at toe of back slope. However, when rain fall intensity is strong (at 400 hours from the start of strong flood period), local hydraulic gradient exceeds 0.5 and shows 0.911 closing to critical hydraulic gradient 0.95. Therefore, embankment at Chandpur cannot be assessed being completely free from piping.

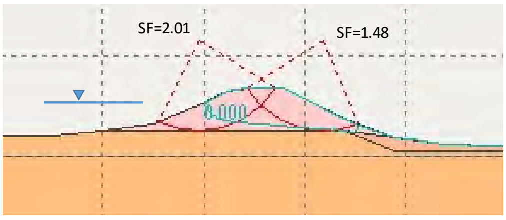
Table 3.2.1Assessment on Slope Stability and Seepage Safety (Chandpur)

<table>
<tr><td>Water level

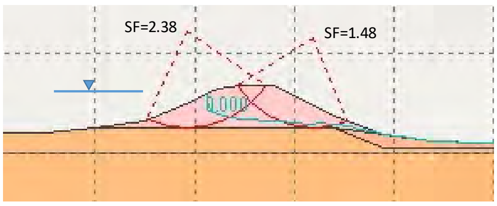
Rain fall intensity</td><td colspan="4">
</td></tr>
<tr><td colspan="2" rowspan="2">Potentila Circular Sliding and Seepage Line</td><td colspan="2">SF(Safety Factor of Slope Stability)</td><td>Local Hydraulic Gradient(Back Slope on Country Side)</td></tr>
<tr><td>Back Slope</td><td>Front Slope</td><td>Toe of Back Slope</td></tr>
<tr><td>1

(0hour)</td><td>
</td><td>1.477</td><td>1.855</td><td>Horizontal
0.114

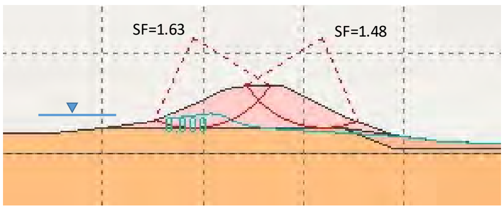
Vertical
0.005</td></tr>
<tr><td>2

(400hours)</td><td>
</td><td>1.476</td><td>2.009</td><td>Horizontal
0.348

Vertical
0.911</td></tr>
<tr><td>3

(600hours)</td><td>
</td><td>1.476</td><td>2.375</td><td>Horizontal
0.166

Vertical
0.052</td></tr>
<tr><td>4

(1100hours)</td><td>
</td><td>1.477</td><td>1.634</td><td>Horizontal
0.077

Vertical
0.004</td></tr>
<tr><td colspan="5">Critical Hydraulic Gradient ic=(Gs−1.0)/(1.0+e) (Chandpur :0.95)</td></tr>
</table>

3.3 Khulna (Embankment：Clayey Silt, Underground：Clayey Silt)

From Table 3.3.1, followings are clarified.

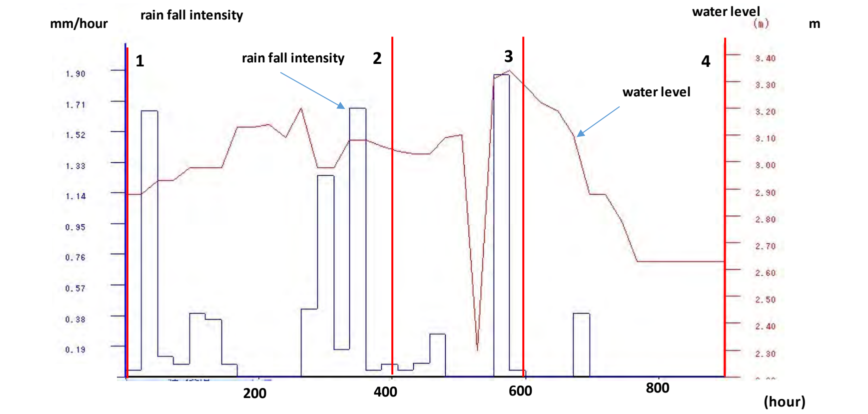
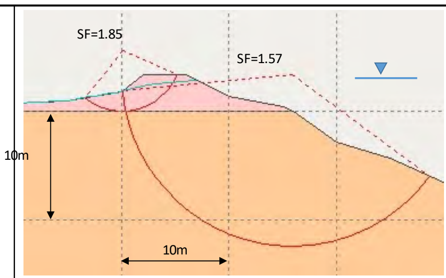
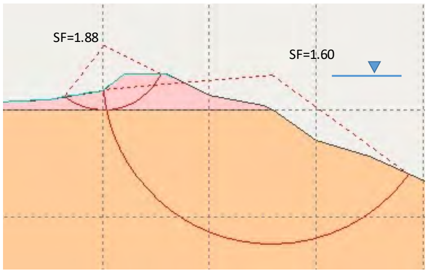
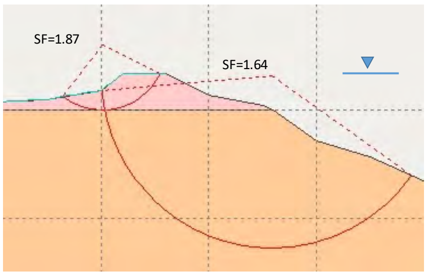
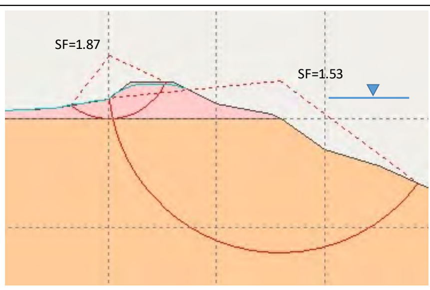
Slope Stability

(1) There are two potential circular slidings, that at back slope on country side of embankment and that at front slope on river side. Among these two, the circular sliding with minimum safety factor is that at front slope on river side.
This seems due to the fact that the edge of embankment is near close to the river side having deep water depth. This is similar to the embankment at Bogra.
In this situation, the safety factor is over 1.5. Therefore, slope stability is assessed to be secured.

Seepage Safety

(1) Seepage line moves closer toward the toe of embankment, emerging on the surface at near the toe of embankment when rain fall intensity is small, at the start and at the end of strong flood period.
(2) When rain fall intensity is strong, seepage line reaches to the toe of back slope of embankment after proceeding on surface of the top of embankment and back slope on country side. It may be due to the fact that rain fall and seepage from the river side slowly moves toward the underground of embankment with allowing storaging themselves into the embankment due to very low permeability of the embankment.
(3) Local hydraulic gradients at the toe of back slope are less than critical hydraulic gradient. The local hydraulic gradient slightly exceeds 0.5 at several times during strong flood period. However, the embankment at Khulna is dominantly clayey silt with its low permeability. Therefore, piping due to seepage seems unlikely to occur. Seepage safety seems to be secured.

Table 3.3.1 Assessment on Slope Stability and Seepage Safety (Khulna)

Water level Rain fall intensity				
	Potential Circular Sliding and Seepage Line	SF(Safety Factor of Slope Stability) Back Slope	SF(Safety Factor of Slope Stability) Front Slope	Local Hydraulic Gradient(Back Slope on Country Side) Toe of Back Slope
1 (0hour)		1.848	1.566	Horizontal 0.249 Vertical 0.065
2 (400hours)		1.875	1.601	Horizontal 0.642 Vertical 0.212
3 (600hours)		1.874	1.642	Horizontal 0.652 Vertical 0.222
4 (900hours)		1.867	1.526	Horizontal 0.602 Vertical 0.179

Critical Hydraulic Gradient ic=(Gs−1.0)/(1.0+e) (Khulna :0.90)

3.4 Moulvibazar (Embankment：Clayey Silt, Underground：Clayey Silt)

From Table 3.4.1, followings are clarified.

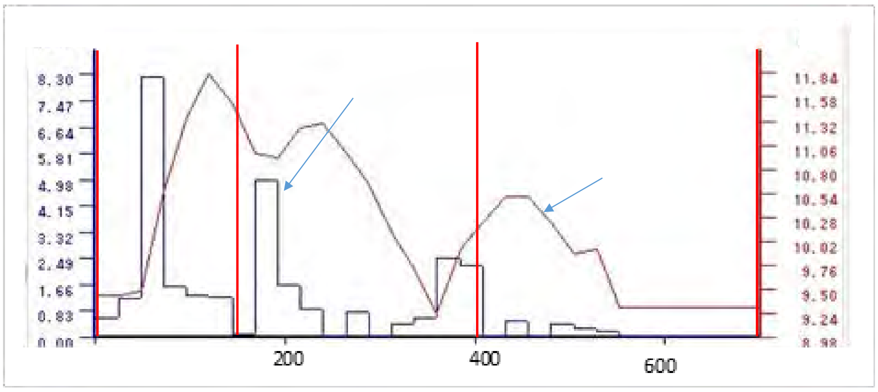
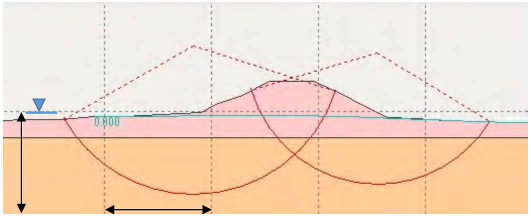
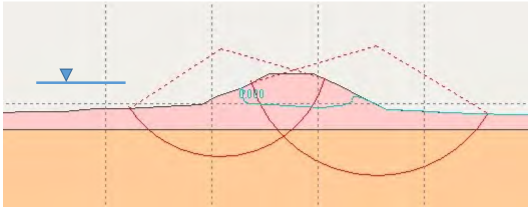
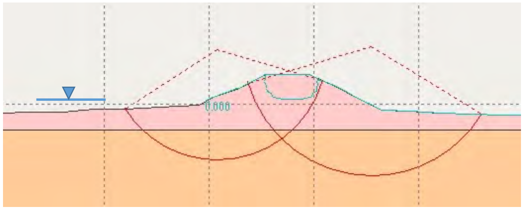
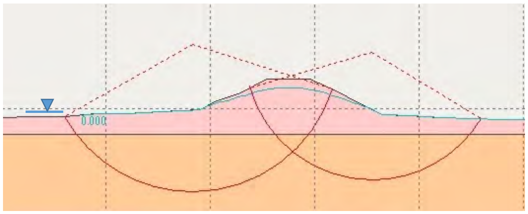
Slope Stability

(1) There are two potential circular slidings, that at back slope on country side and that at front slope on river side. Among these two, the circular sliding with minimum safety factor is that at back slope on country side.
The safety factor is over 2.0. Slope stability is assessed to be secured.

Seepage Safety

(1) Seepage line moves closer toward the toe of embankment, emerging on the surface at near the toe of embankment.
(2) Local hydraulic gradients at the toe of embankment are less than critical hydraulic gradient and at the same time less than 0.5. Therefore, piping due to seepage seems unlikely to occur. Seepage safety seems to be secured.

Table 3.4.1 Assessment on Slope Stability and Seepage Safety (Moulvibazar)

Water level Rain fall intensity				
Potential Circular Sliding and Seepage Line		SF(Safety Factor of Slope Stability)		Local Hydraulic Gradient(Back Slope on Country Side)
		Back Slope	Front Slope	Toe of Back Slope
1 (0hour)		2.227	2.564	Horizontal 0.040 Vertical 0.015
2 (150hours)		2.224	3.57	Horizontal 0.496 Vertical 0.198
3 (400hours)		2.209	2.761	Horizontal 0.496 Vertical 0.291
4 (700hours)		2.204	2.521	Horizontal 0.496 Vertical 0.225

Critical Hydraulic Gradient ic=(Gs-1.0)/(1.0+e) (Moulvibazar :0.78)

4. Conclusion(Slope Stability and Seepage Safety of Embankments)

In this report, slope stability and seepage safety of existing embankments which are subject to strong flood period were assessed by seepage flow analysis and slope stability analysis under the condition that embankment is protected against prior incremental erosion due to river currents during entire rainy season. In this assessment, unsteady seepage flow analysis and associated slope stability analysis were adopted in which time dependent actual flood water level and rain fall intensities changing during strong flood period can be reasonably expressed.

Table 4.1 shows minimum safety factors of slope stability of embankments at Bogra, Chandpur, Khulna, and Moulvibazar obtained from unsteady seepage flow analysis. .

Table 4.2 shows local hydraulic gradient both at the toe of embankment and at nearby surface of underground of the embankments at Bogra, Chandpir, Khulna, and Moulvibazar.

From these assessments, followings are clarified.

1. Slope stability of embankments during strong flood period under the condition that the embankments are protected against prior incremental erosion

The minimum safety factor of slope stability during strong flood period is over or around 1.5 on all embankments. Therefore, slope stability of existing embankments during strong flood period is assessed to be secured if the embankment is protected against prior incremental erosion due to river current.

Bogra: Back Slope 4.39, Front Slope 2.23 Chandpur:Back Slope 1.48,Front Slope 1.63
Khulna:Back Slope 1.87, Front Slope1.53 Moulvibazar:Back Slope 2.20, Front Slope 2.52

2. Seepage safety of embankments during strong flood period under the condition that the embankments are protected against prior incremental erosion.

Seepage line appears on the surface near the toe of embankment.

Local hydraulic gradients are less than critical hydraulic gradient at the toe of back slope at all embankments.

However, at the embankment at Bogra, local hydraulic gradient exceeds 0.5 when water level reaches maximum height. And at the embankment at Chandpur, local hydraulic gradient exceeds 0.5 when rain fall intensity is strong.

Bogra:0.51(at 600hours) Chandpur:0.911(at 400hours)

These embankments at Bogra and Chandpur are categorized Type A or embankments along major rivers. These can be characterized as being built on the foundation ground predominantly thick sandy soil layer and therefore piping at the toe of back slope should be carefully dealt with.

At the embankment at Khulna, local hydraulic gradient slightly exceeds 0.5. However, piping is unlikely to occur because the embankment at Khulna is dominantly clayey silt with low permeability.

Khulna:0.642(at 400hours),0.652(600hours), 0.602(at 900hours)

Seepage safety of embankments at Khulna and Moulvibazar were assessed to be secured.

Table 4.1 Minimum Safety Factor of Slope Stability

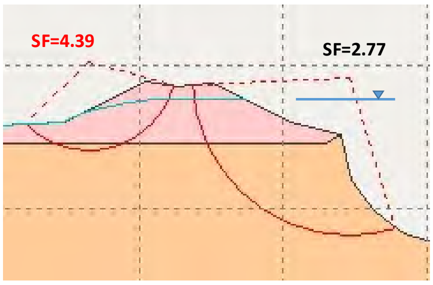
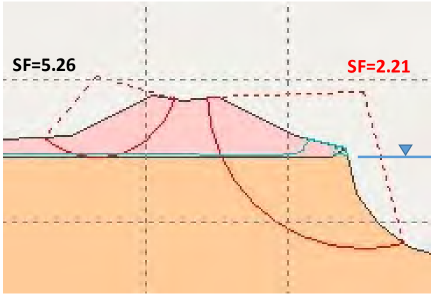
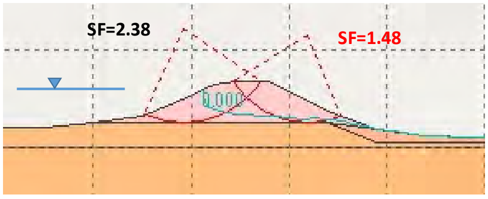
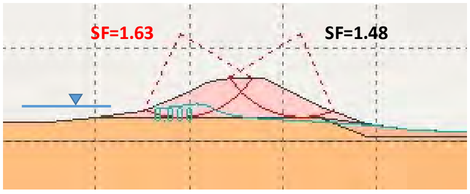
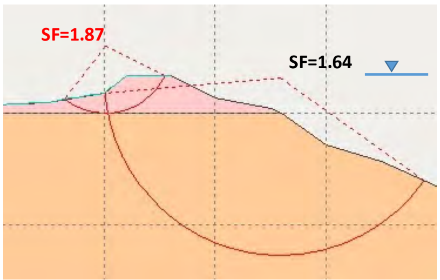
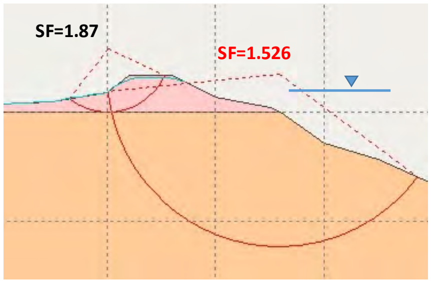
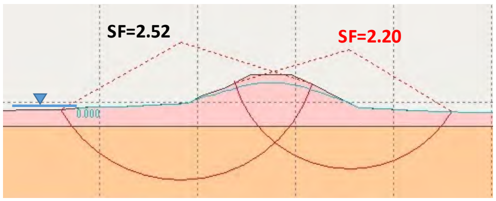
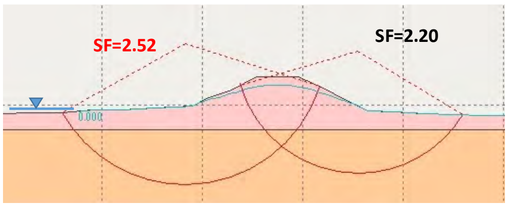
Location	SF(Safety Factor of Slope Stability)	
	Minimum	
	Back Slope	Front Slope
Bogra	 SF=4.388（600 hours）	 SF=2.214（1400 hours）
Chandpur	 SF=1.476（600 hours）	 SF=1.634（1100 hours）
Khulna	 SF=1.874(600 hours）	 SF=1.526(900 hours）
Moulvibazar	 SF=1.204(700 hours）	 SF=2.521(700 hours）

Table4.2 Maximum Local Hydraulic Gradient for Seepage Safety

Location	Local hydraulic Gradient i (Back Slope on Country Side) Maximum Toe of back Slope
Bogra	Horizontal：0.492 Vertical：0.51
Chandpur	Horizontal：0.348 Vertical：0.911
Khulna	Horizontal：0.652 Vertical：0.222
Moulvibazar	Horizontal：0.496 Vertical：0.291

Critical Hydraulic Gradient
Bogra ：0.95
Chandpur ：0.95
Khulna ：0.90
Moulvibazar ：0.78

5 Additional Study (Triggering Factor of Damages of Embankments)

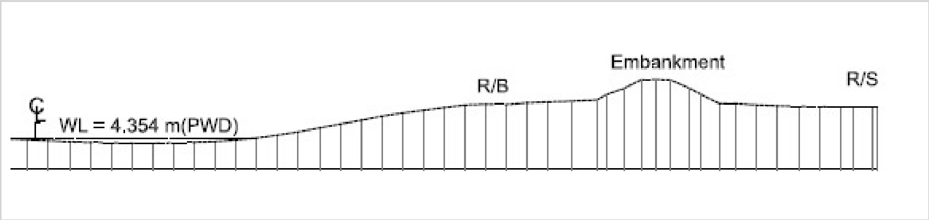
5.1 Damages of Embankments in Bangladesh

From the site survey, it was assumed that damages of embankments in Bangladesh during strong flood period is mainly triggered by prior incremental erosion from the edge of setback in front of embankment which had been subject to river current during entire rainy season. This type of damage was observed in all four types, A,B,C,D.

Among those four types, we obtained the cross section data of damaged part in Moulvibazar and Khulna.

Fig 5.1.1 and Fig 5.1.2 show the cross section of damaged part of embankment. In each figure, sound part of embankment and damaged part of embankment are described.

Sound Part of Embankment at Moulvibazar



Damaged Part of Embankment at Moulvibazar

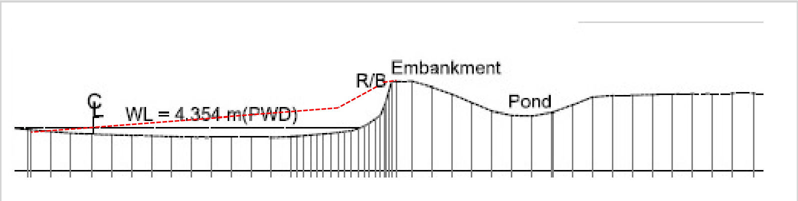
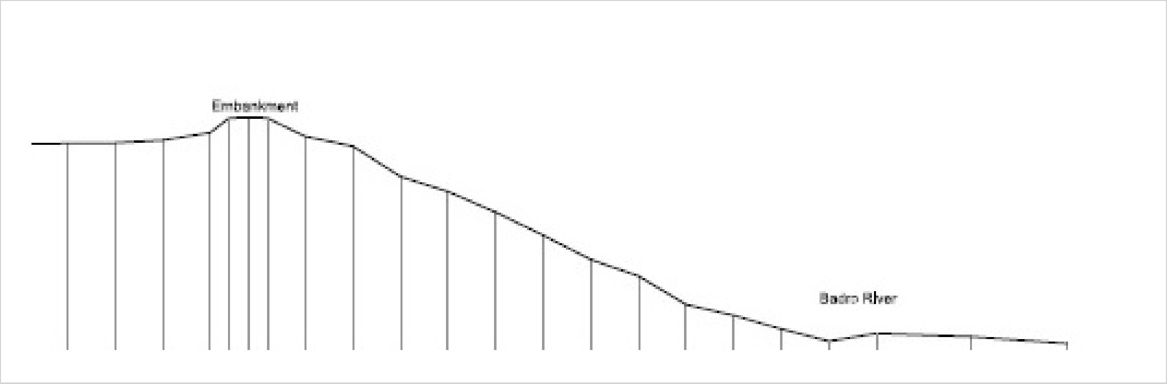


Fig 5.1.1 Damage of Embankment at Moulvibazar

Sound Part of Embankment at Khulna



Damaged Part of Embankment at Khulna

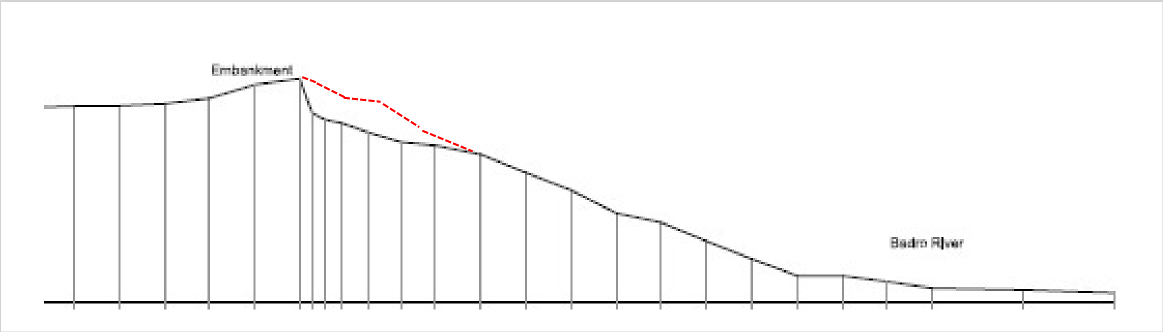


Fig 5.1.2 Damage of Embankment at Khulna

5.2 Impact of Incremental Erosion by River Current

In the previous chapters in this report, it was clarified that Slope Stability and Seepage Safety are secured during strong flood period if the embankment is protected against prior incremental erosion due to river current during entire rainy season.

In this section, slope stability of embankment during strong flood period which had been subject to incremental erosion was investigated with unsteady seepage analysis and associated slope stability analysis. In this analysis, part of setback in front of sound embankment was pre-eliminated toward the front slope on river side of the embankment to simulate the prior incremental erosion.

With this analysis, impact of incremental erosion on damages of embankments in the past was investigated.

5.2.1 Impact of Incremental Erosion on the damage of embankment at Moulvibazar

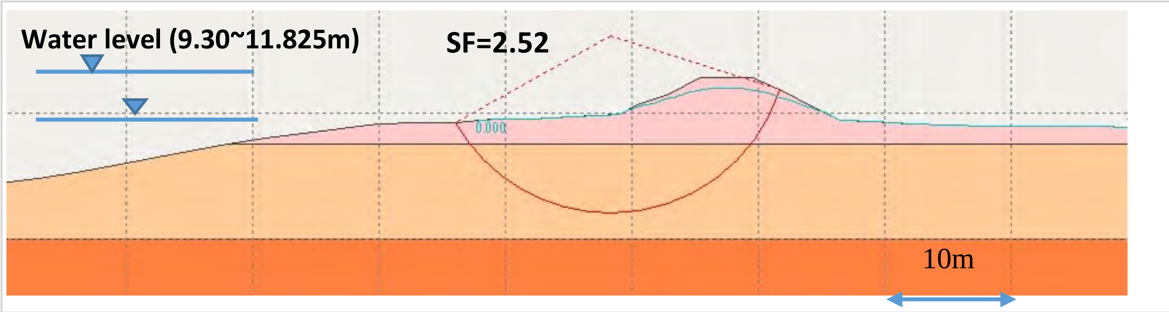
Slope stability during strong flood period was studied on the Moulvibazar embankment which had already suffered prior incremental erosion due to river current during entire rainy season

Prior to this study, in the simulation model, setback in front of sound embankment was eliminated toward the front slope on river side of the embankment. On that simulation model, unsteady seepage flow analysis and associated slope stability analysis were conducted.

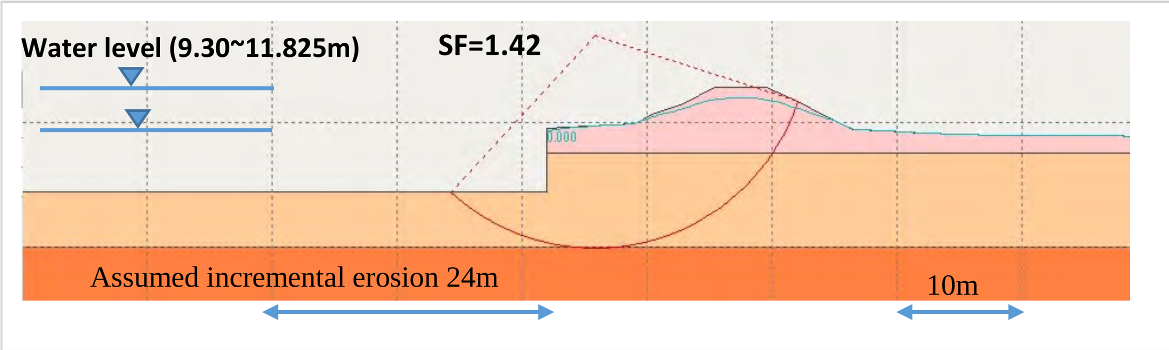
Fig 5.2.1.1 shows the potential circular sliding, minimum safety factor and seepage line when the safety factor shows its minimum value on the embankment which had not suffered prior incremental erosion ,on the embankment in which incremental erosion of 24.0m is assumed, and on the embankment in which progressed incremental erosion of 31.0m is assumed.

In this Fig5.2.1.1, the shape of damaged part of embankment is also described along with above mentioned simulation results.

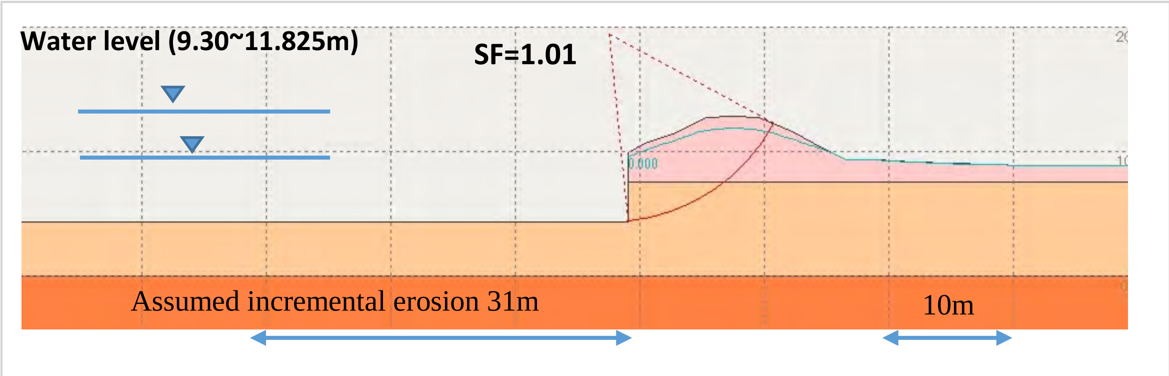
From Fig 5.2.1.1, it is clarified that safety factor declined with the progress of incremental erosion.



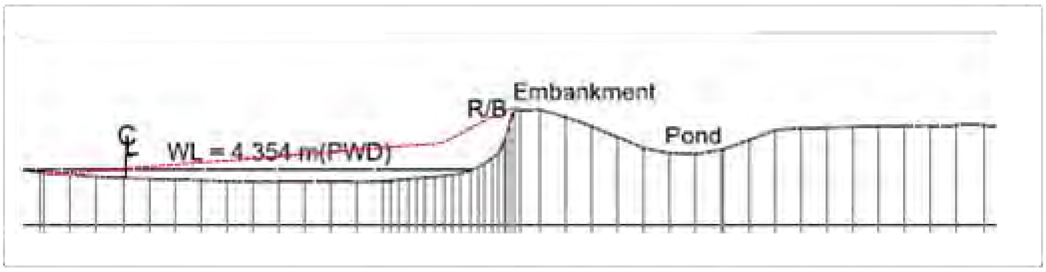
Minimum safety Factor of Embankment at Moulvibazar without incremental erosion



Minimum Safety Factor of Embankment at Moulvibazar with Incremental Erosion of 24m



Minimum Safety Factor of Embankment at Moulvibazar with Incremental Erosion of 31m

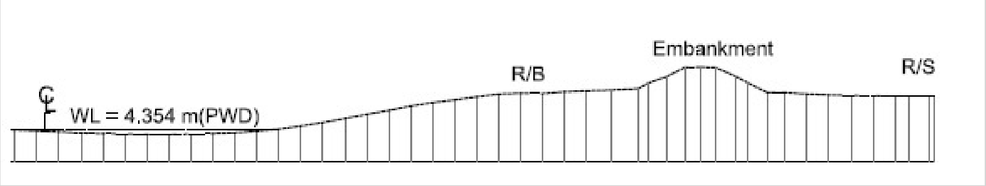


Damaged Part of Embankment at Moulvibazar

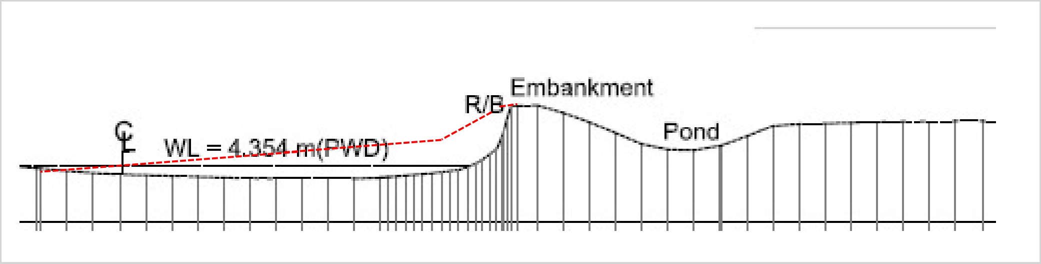
Fig 5.2.1.1 Minimum Safety Factor of Embankment at Moulvibazar with Incremental Erosion

Fig 5.2.1.2 shows the shape of sound embankment and that of nearby damaged part of embankment along with the simulation result..

Sound part of Embankment at Moulvibazar



Damaged Part of Embankment at Moulvibazar



Simulation Result

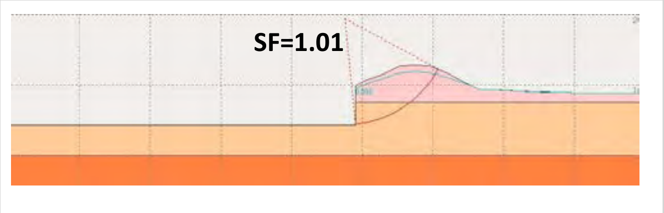


Fig 5.2.1.2 Impact of Incremental Erosion on Damage of Embankment

Fig 5.2.1.2 shows fairly good consistency between the result of slope stability analysis and actual damaged part of embankment.

5.2.2 Impact of Incremental Erosion on the damage of embankment at Khulna

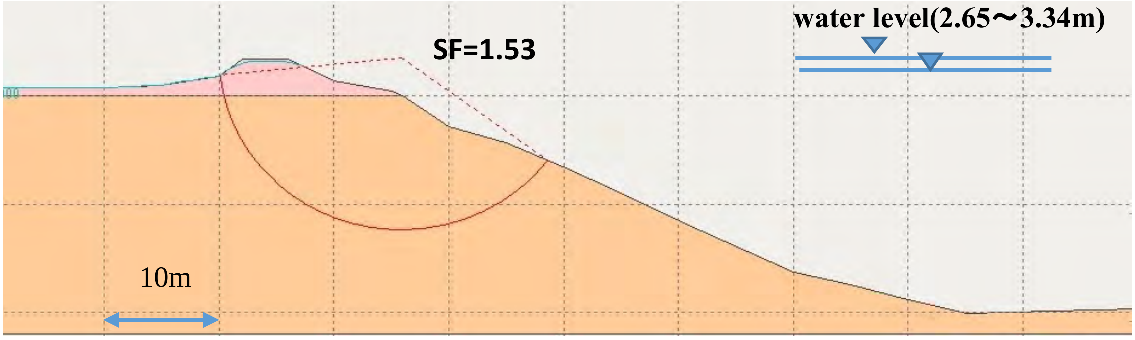
Slope stability during strong flood period was studied on the Khulna embankment which had already suffered prior incremental erosion due to river current during entire rainy season.

Prior to this study, in the simulation model, setback in front of sound embankment was eliminated toward the front slope on river side of the embankment. On that simulation model, unsteady seepage flow analysis and associated slope stability analysis ware conducted.

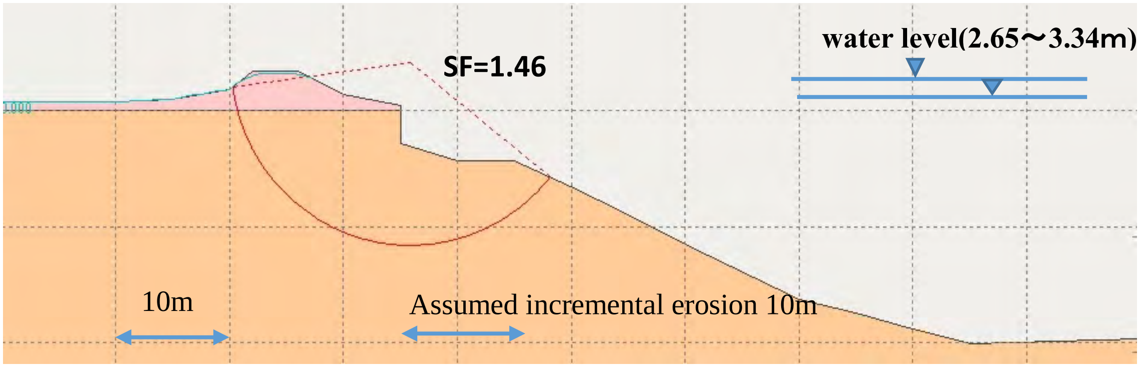
Fig 5.2.2.1 shows the potential circular sliding , minimum safety factor and seepage line when the safety factor shows its minimum value on the embankment which had not suffered prior incremental erosion, on the embankment in which incremental erosion of 10.0m is assumed, and on the embankment in which progressed incremental erosion of 16.0m is assumed.

In this Fig 5.2.2.1, the shape of damaged part of embankment is also described along with above mentioned simulation results.

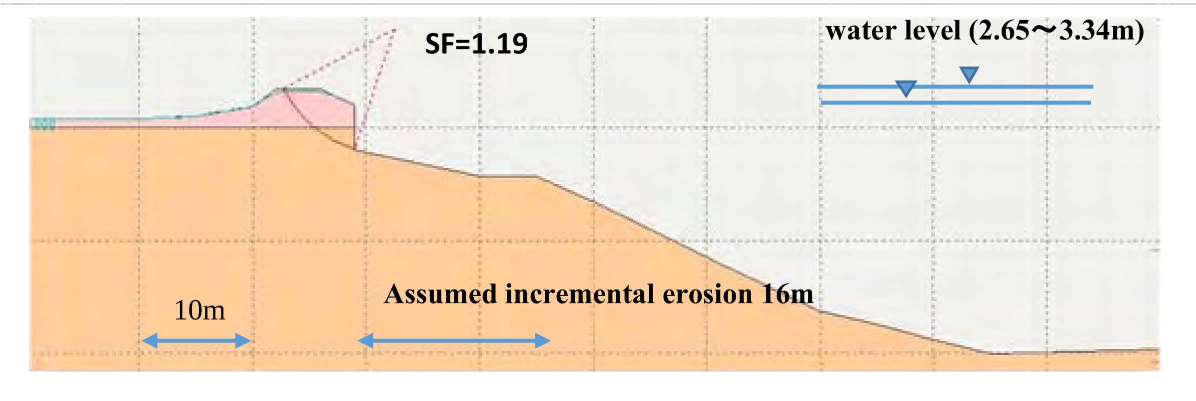
From Fig5.2.2.1, it is clarified that safety factor declined with the progress of incremental erosion.



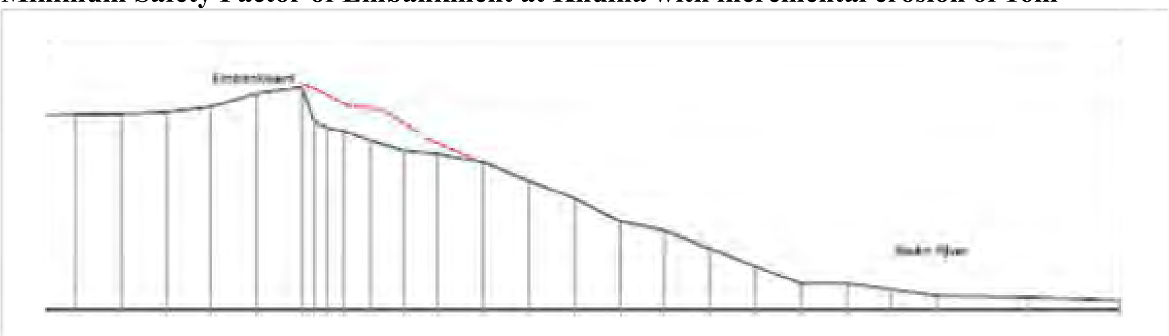
Minimum Safety Factor of Embankment at Khulna without incremental erosion



Minimum Safety Factor of Embankment at Khulna with incremental erosion of 10m



Minimum Safety Factor of Embankment at Khulna with incremental erosion of 16m

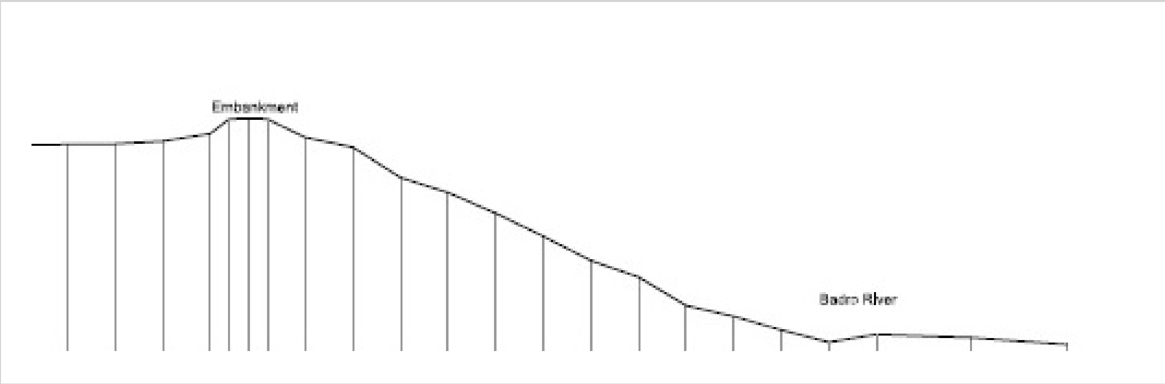


Damaged Part of Embankment at Khulna

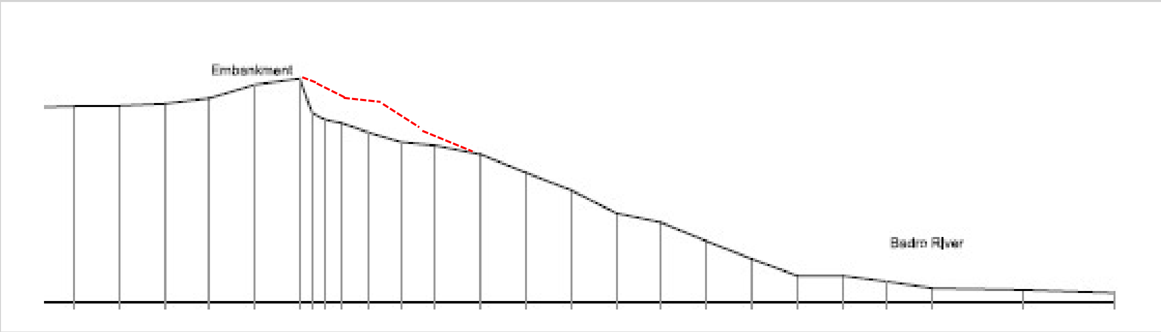
Fig 5.2.2.1 Minimm Safety Factor of Embankment at Khulna with Incremental Erosion

Fig 5.2.2.2 shows the shape of sound embankment and that of nearby damaged part of embankment along with the simulation result.

Sound Part of Embankment at Khulna



Damaged Part of Embankment at Khulna



Simulation Result

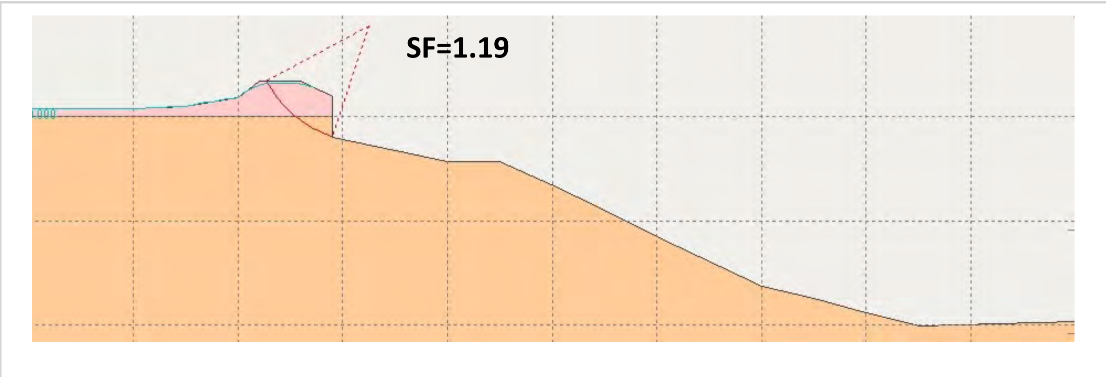


Fig 5.2.2.2 Impact of Incremental Erosion on Damage of Embankment

Fig 5.2.2.2 shows fairly good consistency between the result of slope stability analysis and actual damaged part of embankment.

5.3 Conclusion of Additional Study(Triggering Factor of Damage of Embankments in Bangladesh)

Identification of triggering factor of damage of embankment in the past.

Slope stability and seepage safety of embankments during strong flood period which had suffered prior incremental erosion from the edge of setback in front of embankment due to river current were assessed

From this assessment of embankments, it was clarified that safety of slope stability during strong flood period declines due to prior incremental erosion at the setback of embankment and this prior incremental erosion may be one of major triggering factors of damages of embankments in the past.

ANNEX 2:
Construction Manual for River Embankment

The Project for Capacity Development of Management for Sustainable Water Related Infrastructure
In
The People's Republic of Bangladesh

Construction Manual for River Embankment

Bangladesh Water Development Board: BWDB
and
JICA Expert Team (IDEA/Ingerosec/ESS)

September 2017

Table of Contents

Introduction of Examples in Pilot Repair Works;

Preface - Background of Draft of Construction Manual for River Embankment –

Section 1 Objective of the Construction Manual

Bangladesh is located in the lower reaches of three international rivers: the Ganges, Jamuna and Meghna. With approximately 80 percent of its land in low-lying floodplains, the country is susceptible to inundation during monsoon season. Also, cyclones are hitting Bay of Bengal causing serious damage to the coastal areas and low-lying lands. Factors including these weather and topographical conditions, as well as changes in social climate from population increases and results into slow economic growth, cause much economic slowdown.

The Bangladesh Water Development Board (BWDB) constructs different types/kinds of embankment in different areas/rivers contexts and in different flooding situations such as flood plain areas, sea areas, hilly areas, haor areas, flash flood areas, irrigation projects areas, urban and town areas and major/médium/minor rivers. BWDB constructs, operates and maintains flood embankment and other facilities to alleviate damage from flooding in Bangladesh, but flood embankment failures and other issues result in repeated damage, causing asset loss as well as displacing many people time and again.

Flood control embankment construction work in Bangladesh is performed in conformance with the Technical Specification for Civil Works of BWDB (TS of BWDB) and the Standard Schedule of Rates Manual (SSoRM of BWDB). But something more is required to be addressed about preparatory survey, detailed quality control and safety measures to implement the Works.

Thus, it is required to draft a Construction Manual that supplements the said TS and SSoRM and effects the improvement of river embankment in Bangladesh. Such Manual describes the work items of supervision for Quality Control or Safety management with consideration of status of Bangladeshi natural condition and circumstances of construction field. The Objective of proposed Construction Manual is to increase the capability of BWDB, who are in charge of implementation, so that the improvement in respect of the supervision for river embankment construction will be expected.

Section 2 Usage of the Construction Manual

The Construction Manual for River Embankment clearly states planning and control methods and various notes for construction and quality management, and also actual compaction degree of embankment works under the construction of River Embankment Works. It is possible for a procurement entity to utilize concrete contents of the manual when they draft Tender Document including technical specifications.

As the Construction Manual for River Embankment also includes items which the preliminary study and construction labor safety measures must be considered as a checkpoint of execution of the river embankment project, BWDB can utilize these items in construction supervision. In particular, this Manual is desired to use in quality control for the embankment body. Because it gives concrete figures for quality characteristics such as fill material granular distribution (grading of soil), degree of compaction etc. and shows actual frequency of quality control testing and tentative easy testing method on site, it will be useful.

Section 3 Responsibilities and Obligations of Contractors

3.01 General

Almost all the works of BWDB are executed through contractors. As such the contractors play a vital role in the Works of BWDB to achieve its goal. It is, accordingly, desirable that the engineers of BWDB shall be acquainted with the responsibilities and obligations of the contractors. Similarly the contractors shall also know their obligations and responsibilities in the execution of a work also it is expected that such contractors can utilize this Construction Manual.

In the following paragraphs have been described the responsibilities and obligations of a contractor. This list is, however, not exhaustive. Additional terms and conditions may be added while issuing notice inviting tender if considered necessary.

3.02 Responsibilities and obligations of contractors:

The contractors shall execute the Work according to the drawing and specification and is bound by the terms and conditions of the contract and time limit. He shall, in case he does not normally stay at the site, appoint an agent whose name should intimated to the Engineer in charge or the Project Manager, who are so entrusted by Procuring Entities, in writing. This agent will stay at site and receive, on behalf of the contractor, all orders from the Engineer in charge or the Project Manager and other superior officers and will be responsible for proper execution of the Work. The ultimate responsibilities will, however, rest on the contractor.

The contractors shall keep at the Work site a copy of all drawings and specifications and shall, at all times, give the departmental inspecting officers access thereto. Anything mentioned in the specifications and not shown on the drawings or shown on the drawings and not mentioned in the specification shall be of like effect as if shown or mentioned in both. In case of difference between drawings and specifications the specification shall govern. The decision of Superintending Engineer shall be final, binding and conclusive on all questions relating to meaning of the specification. Time being deemed to be the essence of the contract on the part of the contractor, the contractor must, therefore, arrange his programme of Work to suit the time available and progress must be proportional to time spent, i.e., half the Work must be completed in half the contract time.

Contractor must, if he desires extension of time for valid reason, apply to the Project Manager, within 30 days of the date of hindrance.

Contractor must make over contract Work on completion in a completely finished, clean and fair state including removal of all apparatus and implements required for construction purposes, etc., before final certificate can be given.

Contractor must use stores and materials supplied by the Project Manager at prices specified for his convenience in contract in case the material is supplied by department or Engineer in charge. Such stores become the property of the contractor.

Contractor must accept alteration to and additions in original specifications, drawings, design and instructions if so required by the Project Manager or the Engineer in charge.

Contractor must accept curtailment or alteration or complete cessation of contract if so ordered by the Project Manager or the Engineer in charge in writing and shall not be entitled to any compensation for such change or restriction.

Contractor must, on demand by the Project Manager or his subordinate, remove or rebuilt any defect or unsound workmanship or any other thing not in accordance with the contract at his expense.

Contractor must give at least 5 days' notice in writing of his intention to cover up foundations, or otherwise inaccessible portions, of a structure.

Contractor is liable for all damage or injury to building, road, fence, enclosure or grass land, etc., caused by his servants or work people. He must also put right all defects or any kind which become apparent within the time specified in the contract.

Contractor must supply all materials, plant, tool, appliances, scaffolding, etc., and carry out all temporary works required for proper execution of Work. He must also supply the necessary personnel with means and materials for setting out Works and for counting, weighing and assisting in the measurement or examination of works and materials. He must also provide all fencing and lights to protect the public from hurt or accident and is bound to meet the cost of any legal proceedings brought for damage or injury sustained owing to neglect of such precaution.

He shall not employ female labour within the limits of Cantonment.

He shall employ no labourer under 12 years of age.

He shall not pay his labourers less than the local rates.

He shall pay compensation for injury or death of his work-man or employee in conformity with Workmen's Compensation Act or on the same scale as laid down therein in case payment is made under the Civil Service Regulations.

Contractor may not assign or sub-let without written approval of the Project Manager. He is liable to the severest penalties if he —

(a) assigns his contract or attempts to do so,

(b) becomes insolvent, commences insolvency proceedings or makes composition with his creditors or attempts to do so,

(c) offers bribe, gratuity, gift, loan, perquisite, reward or advantage, pecuniary or otherwise, to any Government officer,

(d) permits such officer or person to become directory or indirectly interested in the contract.

Contractor must refund or permit reduction from any moneys due to him for any excess payment made inadvertently or otherwise and found to be due to the Government.

Contractor must notify the Project Manager of any change of constitution of the firm.

Contractor must accept the decision of Superintending Engineer as final and conclusion on all matters relating to the contract.

Contractor must, if no specification exists either in the contract or in the departmental specifications, accept the instructions and requirements of the Project Manager as to the carrying out the Work.

Contractor must apply in writing for extension for time if his work or supply has been hindered for good reason.

Contractor must remove all rejected materials and place and stack in such position as required by the Project Manager or his Assistant away from all accepted materials before receipt or final certificate.

Contractor shall supply only best quality materials and shall only receive payment for such materials as is passed by the Project Manager or his Assistant.

The contractor shall at his own cost provide his labour with huttings on an approved site and shall make arrangements for water supply, conservancy and sanitation in the labour camp to the satisfaction of the local public health and medical authorities,

Section 4 Importance of Progress Control

It is without saying that it is important of planning and implementation of the works to complete the project as per planned time schedule for the benefit of the recipiant people and society.

But there are found some projects were delay and/or suspended due to unavoidable situation and other reason, resulting with not a small losses or damages to the society.

To avoid such event, it is required; i) well study and good planning, ii) proper time management and iii) taking counter-measures for such events.

This Manual is expressed for such issues in Section 1.2 Construction Plan and Section 1.3 Progress Control for your referenece.

Chapter 1 General items of Supervision for construction (100 General)

Section 1.1 Surveys for Existing Condition

Surveys to be conducted before the construction work include a Climate survey, soil survey, environmental study, work site investigation, and loading/unloading site investigations for the borrow pit and disposal of soil during construction. In order for the work to go smoothly, it is essential that work details are surveyed appropriately and that the concerned parties are familiar enough considering the actual state of the work site.

1.1.1 Natural Condition Surveys

Other than soil survey stipulated in Designing, items and contents for natural condition surveys around the proposed site such as climate surveys are shown on following table;

Table 1.1.1 Items and contents for natural condition surveys

Items of survey	Contents of survey
Rain fall	Intensity of rain fall, numbers of rainy day, quantity of rain fall at each month
Wind	Direction, velocity
Climate	Weather, temperature, hours of sunshine
Hydrology	Average and high water levels, low water level, flood duration, frequency of flood, velocity of water

1.1.2 Site investigations

It is necessary to carry out following investigations in and around the construction site more over natural condition surveys.

Table 1.1.2 Items of investigations in and around the construction site

Investigation for obstacles and buried objects within the work zone
Alignments and right-of-way of other infrastructure, road and railway etc., which may interfering the proposed riverbank alignment
Investigation for historic sites and buried cultural assets
Checking for access road or temporary construction road

Borrow pit and dumping area survey
Status surveys of currently cultivated farmland and residents

Investigation for obstacles and buried objects

It is necessary to discuss with the parties who are in charge of such expecting obstacles and buried objects regarding replacement or treatment measures. Proposed survey items and outline of discussion with owner or administrator of which are as below table;

Table 1.1.3 Summary of consultations on obstacles and solution

Obstacle		Consulting Party (Owner or Administrator)	Outline of Discussion and Solution
House or other building and farm land or other private land		Owner or user	o Method of protective measures o Timing for relocation or removal and its compensation cost
			○ If necessary, acquiring or borrow such building and/or land
Steel towers / utility poles	Power / electricity lines	Power Grid Company of Bangladesh, Ltd. (PGCB; formerly PDB)	o Timing for relocation or removal o For reinforcement, consult with the contractors on construction period and methods
	Communication lines	Govt T&T Dept. Private carriers (Grameen Phone, Banla Link, Airtel, Rabi, etc.)	
Graves		Family who owns the grave	o Time, site and method for relocation (local customs) o Procedures prescribed by law
Buried objects	Water pipelines	Water supply authorities as Public health authorities and WASA	o Time, method and site for relocation or removal o For reinforcement,

<table>
<tr><td rowspan="4"></td><td>Sewerage lines</td><td>WASA, BWDB and RHD</td><td rowspan="4">consult the timing and methods
○ Consult on protection method and reach</td></tr>
<tr><td>Communication lines</td><td>T&T dept. and private carriers</td></tr>
<tr><td>Power lines</td><td>PGCB</td></tr>
<tr><td>Gas pipes</td><td>Titas Gas</td></tr>
<tr><td rowspan="3">Roads</td><td>Highways</td><td>Roads and Highways Dept. (RHD)</td><td rowspan="6">○ Checking the overlap of proposed river embankment alignment and existing (or proposed) other infrastructures.

○ Time and methods for protection or relocation where the river crosses

○ Consultation on relocation and methods as safety measures

○ Procedures prescribed by laws and ordinances on usage permits, etc.

○ Fixation of alignments for proposed river embankment and other infrastructure. Also making sure right of way of the proposed river embankment.</td></tr>
<tr><td>Local / rural roads</td><td>Local Government Engineering Department (LGED)</td></tr>
<tr><td>City roads</td><td>Respective Municipality</td></tr>
<tr><td colspan="2">Railroads</td><td>Bangladesh Railway</td></tr>
<tr><td rowspan="2">Rivers</td><td>Buried object control within the river or river embankment</td><td>BWDB</td></tr>
<tr><td><Classified by river characteristics (Ex.)>
・River embankment on the 3 major rivers (flood plain areas)
・ Southern seawall (sea areas)
・Inland river embankment
1) Polder
2) flash flood embankment
3) Haor Submersible embankments
4) Waterway river bank (irrigation projects) and etc.</td><td>Flood measure agency is BWDB, except for small structures

ADG of Eastern & Western Regions

8 O&M Circle zones

Each division: O&M Division Office</td></tr>
</table>

If necessary for BWDB to obtain the land owned by others in order to execute riverbank project, BWDB is to acquire such land or compensate the cost for temporary use of such land.

1.1.3 Status surveys of farmland and residents within the scheduled construction area

If there are people currently residing, actively farming or otherwise using the area scheduled for river embankment construction, current status of the area must be surveyed before work commences. After field reconnaissance, including surveying to clarify current location relationships, the following are to be studied by interview or other available means, and consultation document prepared:

(1) Owners and users of farmland and residences
(2) Drawings given the location of farmland and buildings related to the proposed construction.
(3) Work schedule

Following the survey, the owners, users, contractors and any other stakeholders in the river embankment project are to consult about moving farm holdings or residences. If necessary to obtain such farmland as permanent river bank, BWDB may acquire it for project from land owner.

Mutual agreement is to be made about measures to be taken. As such consultation and measures may require significant time. And sufficient time must be allowed for preliminary studies.

Section 1.2 Construction Plan

1.2.1 General

Objective of Construction Plan is to draft the construction method and the construction procedure in order to execute the river embankment with the structural shape and quality defined by the designing documents. For drawing up the Construction Plan, it is necessary to consider various factors such as natural conditions, social restriction, scale of budget and status of access to the site more over the requirement of design.

1.2.2 Drafting the Basic Construction Plan

The construction plan includes absolutely every construction-related item.

The key items are given as follows:

a) Soil movement plan (cut/fill)
b) Construction methods for each category of work, a usage plan for the requisite construction machinery, construction pace and required periods
c) Work sequences for each category of work, construction periods and the overall work schedule progress
d) Labor and material plans
e) Work site organization and temporary facilities plan
f) Plans for access roads and other preparatory work
g) Plans for accident prevention and health and safety
h) Conservation plan for the local environment

The general procedure for preparing a construction plan is given in Figure 2.2.1. The progress is outlined step by step in the items below.

1) Collection of information

Collect and sort through the requisite data for preparing the construction plan. Study the design drawings and survey site conditions, including natural and social conditions.

2) Soil movement

Study the terrain, design height, borrow pit, locations where soil will be loaded and unloaded, and other relevant details, and create a rational soil movement plan. Based on this plan, specify transport distances and work out details for soil movement.

3) Establish a work sequence and mark work sections

Differentiate work sections according to soil movement status and structure locations. Next, prioritize the sections and key work categories, and then establish a work sequence. During this time, consider the necessity of any temporary or preparatory work, such as access roads.

4) Examination for construction methods and work schedule progress

Construction methods and machinery for the key work categories are to be selected and comparison of work costs are to be found. Based on the comparisons, calculate the days needed for each work category. While considering work durations and sequences, arrange the works so that they will fit into the construction period, finally establishing a work schedule progress.

5) Conduct overall assessments

Comprehensively evaluate all the alternatives based on the construction period, cost and other conditions, and then select the plan that meets each condition. If no alternative satisfy all the construction restrictions and conditions, the previous step are to be examined and an appropriate plan are to be found out.

6) Finalization of construction plan

Once the construction plan is finalized for the key work categories, a detailed plan for each item should be prepared while checking for inconsistencies and reviewing the work management plan.

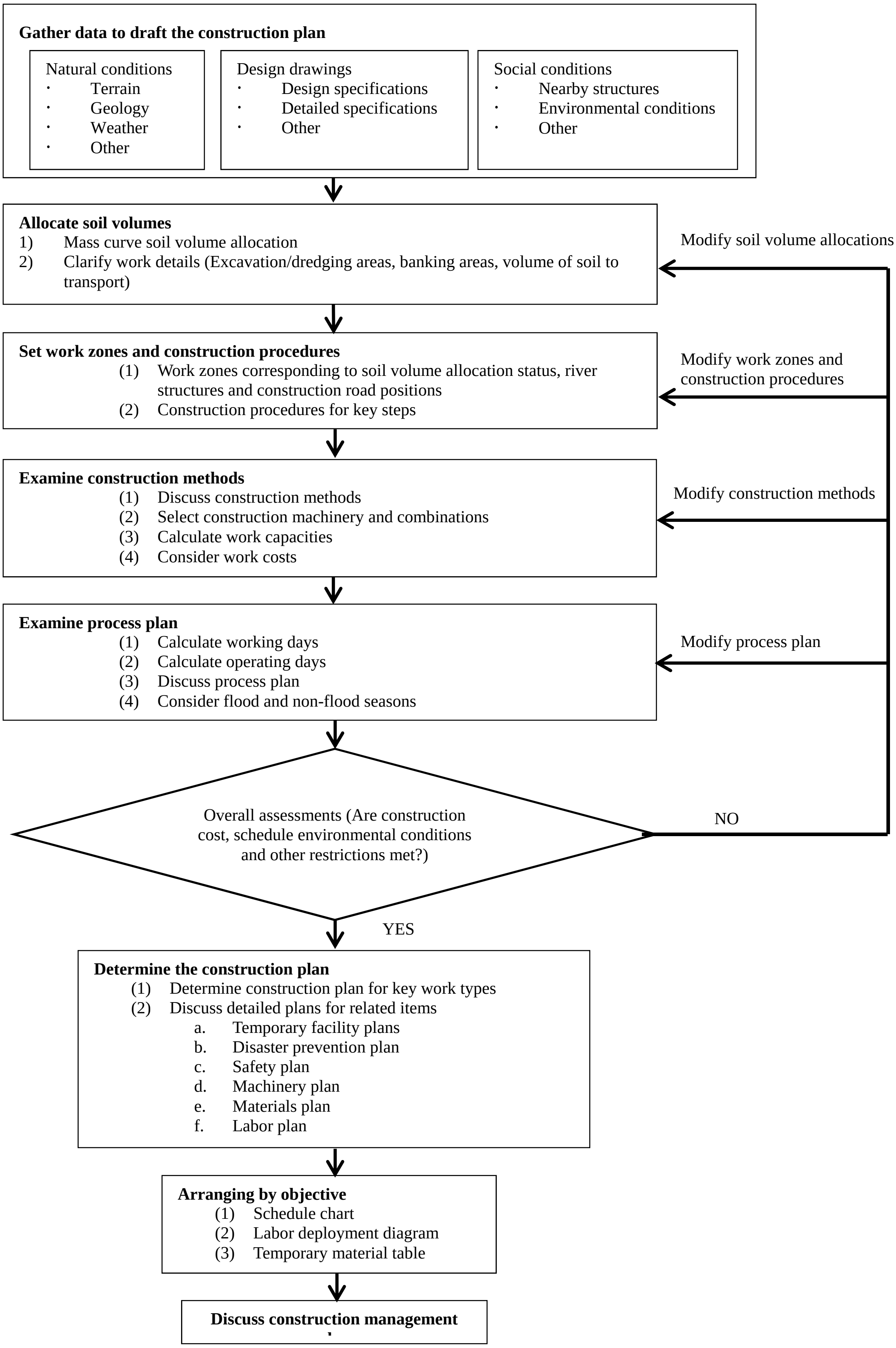


Figure 1.2.1 Procedure for drafting the construction plan

Section 1.3 Progress Control

1.3.1 Progress Plan

Objective of drafting for construction progress plan is given as follows:

a) To decide the order to implement each progress or partial task.
b) To determine the appropriate actual working period for each progress or partial task and
c) To prepare a schedule that contains all work within the construction period and each progress or partial task within a reasonable time.

More over setting the period for preparatory and temporary works, drafting procedure and progress plans will be carried out as below;

1) Preparing the progress plan

First, formulate the schedule for earthworks and other types of work by determining the combination of machinery to be used, number of workable days and daily workloads for each construction method. Next, create a progress plan, looking at the days required for each type of work and order of works.

The progress plan must be determined considering for the volumes of soil filled, excavated, dumped and borrowed, as well as soil quality, terrain, weather, construction period, temporary work and others.

There is a case where river training work must be performed in seasons with adverse conditions. Particularly with earthwork, where weather can dictate work capacity or even feasibility, factors such as rainfall, distribution of rain days, temperatures and hours of daytime should be sufficiently studied. It is also necessary to study river water levels, flow and geological features and consider protection of revetment in order to rationally determine the number of workable days.

(1) Calculating number of working days

The number of working days is the number of days required to perform a certain task. This can be calculated by the formula below. The principle is the same in calculations for earthwork, even for complex work of great volume involving many types of construction machinery.

$$\text{Working days} = \frac{\text{Total earthwork volume, or volume for each site (m}^3\text{)}}{\text{Daily workload (m}^3\text{/day)}}$$

Where:

$$\text{Daily workload } (m^3/\text{day}) = \left[\begin{array}{l} \text{Hourly workload for construction} \\ \text{machinery or combined} \\ \text{construction } (m^3/\text{ hr.}) \end{array} \right] \times \text{Daily operating hours (hrs./day)}$$

Or:

$$= \text{Hourly workload for manual labor } (m^3/\text{hr.}) \times \text{Daily working hours (hrs./day)}$$

If working days as calculated above are used to represent the working days in the schedule chart, calendar days must be calculated according to the percentage of working days in the schedule.

$$\text{Calendar days} = \text{working days} \times \frac{1}{\text{Working days \%}}$$

$$\text{Working days \%} = \frac{\text{Workable days during the work}}{\text{Calendar days during the work}}$$

$$\text{Calendar days during the work} = \text{Workable days} + (\text{days off + rain days + days off after rainfall + other rest days})$$

(2) Percentage of operating days and workable days

Percentage of operating days must be determined the reasonable days on the basis of past weather and hydrological records for the area of the site.

Additionally, for rest days, national public holidays must be put into estimates along with monthly rest days.

Percentage of operating days is usually between 50 and 80 percent, but it is need to consider the decrease by the condition of access road, characteristic of the soil, the scale of the works and other factors.

2) Representation of the progress plan

There are various methods of representation used for progress plans depending on the work scale, importance and other factors. In general, however, working days are calculated for each type of work for use in schedules determined by horizontal line schedule charts, such as bar charts.

Schedule charts must be useful in schedule management. Complex charts may sound good in theory, but may not be practical; over-simplified charts do not allow for the schedule to be managed scientifically.

(1) Horizontal line schedule charts (bar charts, etc.)

Horizontal line schedule charts include bar charts and Gantt charts.

As shown in the example in Figure 1.3.1, bar charts show the steps and working days for each type of work at a glance for an easy-to-understand overall schedule. They divide comprising work by type of work, displaying the working days (or calendar days) needed for each type of work with the work period as the horizontal axis. Meanwhile, Gantt charts, while not used in progress plans, are frequently used in schedule management and are convenient for understanding progress by type of work.

(2) Progress control curve

The progress control curve is a useful tool in comparing the project schedule to the implementation schedule. As given by the solid lines in Figure 1.3.1, the progress control curve expresses the monthly work progress rates with work progress as the vertical axis and work days required as the horizontal axis, where the work progress rate represents the percent of finished work costs for each portion against the overall work costs. Normally, progress control curves are used in combination with a bar chart. As shown in the figure, in a progress plan in which river earthwork progresses smoothly, the progress control curve will be an S-curve, with output increases going from gradual to sharp to gradual again in the early, middle and end stages of the construction period.

(3) Network

The network is a typical systematic approach used in planning and management of construction work (see Figure 1.3.2).

In a network chart, work is divided into a critical path and a float path. The critical path shows how long it will take to complete the work within the construction period with no slack, while the float path involves a certain amount of slack. This chart thus shows how important the start, finish and other progress of each work are, relating the processes to the work as a whole.

Network method has been introduced as a part of the Program Evaluation and Review Technique (PERT). According to the same principle of PERT, Critical Path Method (CPM) is also broadly used for project progress management.

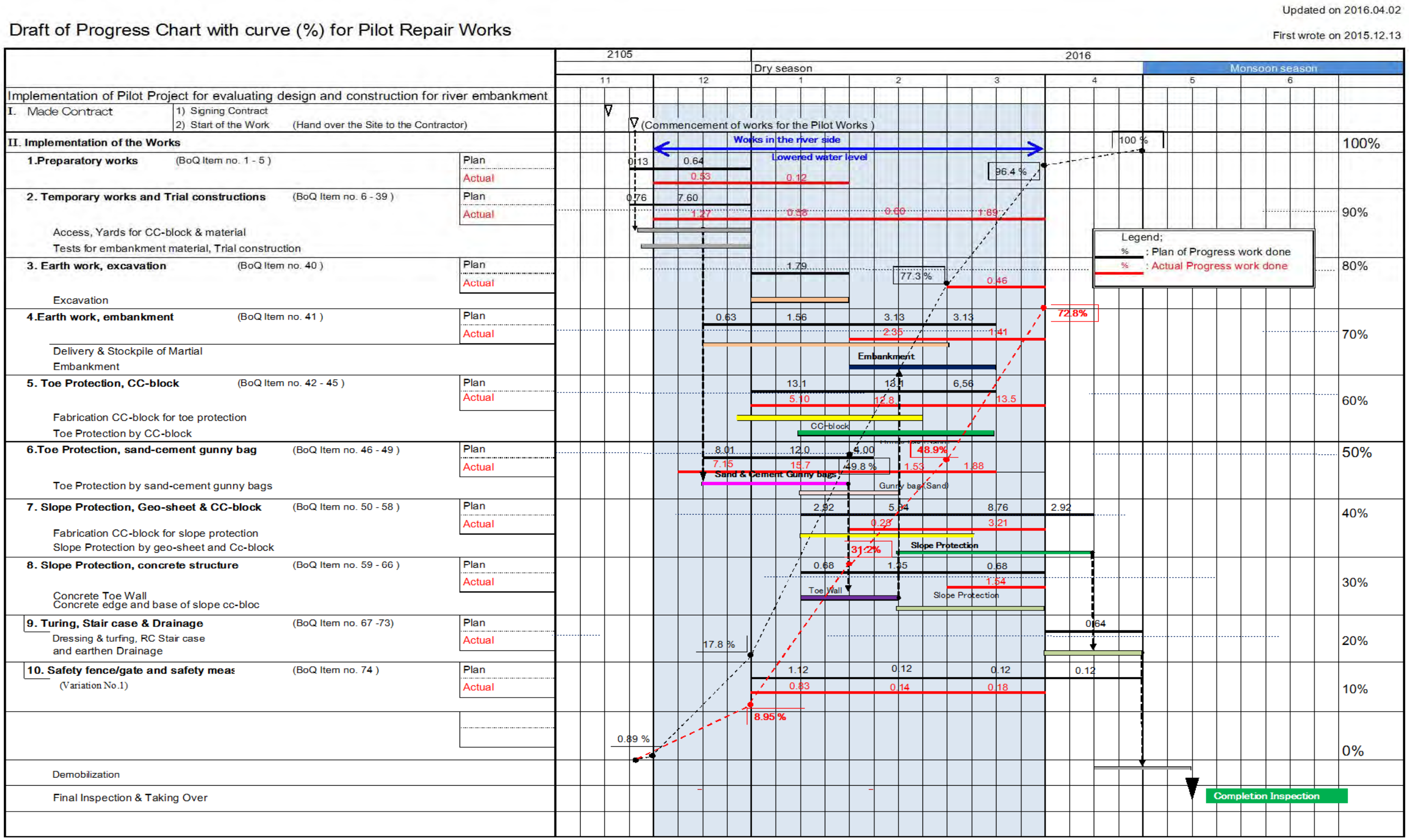


Figure 1.3.1 Horizontal line schedule chart (bar chart) for embankment work

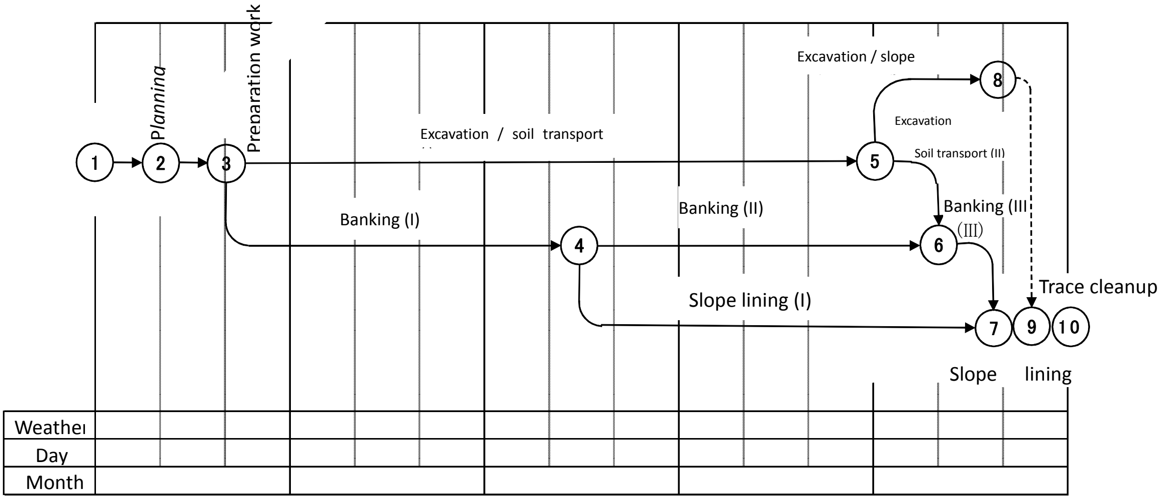


Figure 1.3.2: Sample of network schedule chart

Table 1.3.1 summarizes the strengths and weaknesses of each of the schedule charts outlined above.

Table 1.3.1: Comparison of progress plan charts

	Schematics	Strengths	Weaknesses
Horizontal line schedule charts	(1) Bar chart A B	(1) Easy to read (2) Easy to make (3)Easy to modify	(1) Connection between works unclear (2) Unclear what work affects schedule
	(2) Gantt chart A B	(1) Progress is clear (2) Easy to make chart	(1) Start date, completion date and days needed for each work unclear (2) Connection between works unclear
Cumulative curve schedule charts	(4) Graph schedule chart A B	(1) Schedule is clear (2) Easy to make chart (3) Days required clear	(1) Critical point management work unclear (2) Work relations unclear
	(5) Cumulative progress curve S-curve	(1) Can judge schedule progress (2) Easy to compare plan and actual	(1) Everything but progress unknown

1.3.2 Control for Construction Progress

1) Necessities and Preparation

Schedule management involves managing the work with the work progress so that it is completed efficiently, economically, and within the construction period by following a schedule created based on the construction plan.

In case the progress has changed for some reason, it is important for the actual work to adjust the subsequent progress plan and move ahead with work again based on the new progress plan.

During controlling construction progress, it needs to understand and check the following issues;

Table 1.3.2 Checking items for controlling Construction Progress

Checking items	Contents of checking
(1) Site conditions	- Environmental conditions of the site (topography, geology, metrology and hydrology) - Site conditions such as labor force, materials, and machinery - Coordination of the schedule and implementation with preceding and subsequent work
(2) Contract conditions	- Changes in site shape and work volume - Securing easements, necessity of removal of obstacles and consulting with local parties concerned
(3) Plans for temporary Facilities	- Ensuring the amount and safety of necessary equipment - How to handle changes to site conditions

2) How to Manage the Schedule

(1) Management approach

As stated in the Table1.3.1, schedule management approaches include horizontal line schedule charts (bar chart, Gantt chart), accumulating curve charts (graph schedule chart and cumulative progress curve) and network schedule charts.

The daily workload used in the progress plan is the average workload in normal conditions. However, earthwork may be impeded by weather and other reasons, and slight changes to construction machinery and the labor force may often delay the schedule in the long run. Therefore, it is safer to expect these delays and to make workload during normal conditions is bit larger than the planned average workload.

There are various ways to create schedule charts, but generally a horizontal line schedule chart is used. Figure 1.3.1 shows an example of a schedule chart combining a horizontal line and graph for mainly earthwork.

(2) Daily workloads

The daily workload used in the progress plan is the average workload in normal conditions. River earthworks in particular often do not proceed on schedule due to weather and other causes and may be greatly delayed. Thus, it is safer to anticipate these delays so workload during normal conditions should surpass the planned workload.

Moreover, if workloads are tracked at the site, such records should be used in schedule management. For this reason, it is important to always organize daily work reports and record construction conditions.

(3) Frequency of review to manage the schedule

While there is no definitive rule for determining how frequently to compare the project schedule and actual progress, in principle they should be compared daily. Particularly important review times over the entire construction period are listed below.

Table 1.3.2 Timing of progress management review

Time to review project schedule	Details subject to review
Construction commencement	First review to check if the initially planned temporary facilities and construction-related machinery are ready per schedule, if the main work has commenced, if there are any changes to site conditions and work can proceed under these conditions.
During construction (when work performance is ascertained)	Ascertain the speed of construction work to reassess scheduling for the subsequent work.
When there are any changes to the schedule	When there are any schedule changes, immediately reassess the schedule.

3) Handling Schedule Delays

If any delay to the schedule arises, it is important to ascertain the cause, decide how to deal with it and act quickly.

The following items are to be detailed how to handle delays.

a) Revise work procedures (improve work efficiency)
b) Shift work order
c) Change work times and crew numbers
d) Change the machinery to be used
e) Other considerations

Moreover, rework is a surprisingly frequent and major cause of construction delays. Rework results from careless planning and insufficient management of the work, and therefore care needs to be taken. It is also important to promptly consult with related parties to determine necessary measures for delays that occur due to land issues, disasters, and changes to the shape of the site.

Example of Handing Schedule Delay in a Pilot works;

1. Out-line of the delay

Name of Project: Capacity Development of Management for Sustainable Water Related Infrastructure, BWDB / JICA

Name of Works: The Pilot Repair Works of Manu River Flood Control Embankment in Moulvibazar District

Commencement Date: 25 November 2015

Intended Completion Date: 15 May 2016

It has been extra ordinaly imclemnt weather, such as extraordinary inclement weather on 25th February 2016 and heavy rain fall and its consequent water rise from 28 March 2106. At the middle of May 2016, the water level has been above the bottom level of slope of proposed embankmentand. Under such situation, slope protection works cannot be continued. Thus the Works were suspended during coming monsoon season until next dry season. Intended Completion Date is extended by the 30 April 2017.

2. Anlaysis of the cause of delay

JICA Expert team made a report of the Delay on April 2016 as below;

Report on Progress of Pilot Repair Works (PRW) and Possible Time Extension

26/Apr/2016: JICA Expert Team

1. General

After the rare rainfalls on 28th and 29th of March, the construction works of the pilot repair works (the PRW) has stopped tentatively due to the higher water level at the work site. In this context, the Director of the Design Circle 1, members of the JICA Expert Team and management team of the Contractor of the PRW (T.S.S) discussed the present condition including the possible time extension of the PRW on 19/Apr/2016. This Report presents the summary of Progress and the possible time extension of the PRW, based on the discussion results.

2. Progress of Pilot Repair Works

Progress of the PWR as of 24th April is shown in the following photograph and figures, and are summarized as follows:

Photograph: Overall View of the PRW.
Figure 1(1): Progress of PRW (Plan)
Figure 1(2): Progress of PRW (Representative Cross Section Profile)

Summary of progress (as of 24/Apr/2016)

- Launching apron/riverbed protection by the CC Blocks for the improvement stretches, except transition stretches, was completed 80%.
- Foot protection works by the sand cement mortar bags and the CC Blocks and foundation work (toe wall) of slope protection work for the same stretches were completed.
- Earth works up to the berm level (PWD +9.1m) was completed, but some part of the slope was damaged due to the floods.
- The upper part of earth works, all of slope protection works by the CC Blocks with projection and the transition works to the existing embankment are remained.

3. Hydrological condition around the PRW

In order to grasp the existing hydrological condition around the PRW, the water level data and rainfall data are collected. Locations of the observation sites are shown in Figure 2.

(1) Water Level and Rainfall at Moulvibazar
Data of the water level and Rainfall at Moulvibazar during the dry season (Jan-May) from 2007 to the present are shown in the following table and figure.
Table 1 Summary of monthly rainfall and rainy days (from 2006/2007 to 2015/2017)
Figure 3 Water Level at Moulvibazar (Jan May from 2007 to 2016)

Based on the table and figures, the following matters are identified:

- There was rain with 85mm in Moulvibazar on 25/Feb/2016. There is no such rain at Moulvibazar in February. Due to the rain on 25/Feb/2016, the water level rose up to PWD +8.15 m at Moulvibazar and the PRW could not be conducted for a week.
- It was continuous rain from the end of March of this year. There was no such rain during this period at the Moulvibazar up to last season.

(2) WL at Sherpur on Kushiyara River
Sherpur water level observation station on Kushiyara River, which is the main discharging point of Manu River, is located at about 10 km downstream site of the PRW site. In case of higher water level at Kushiyara River, the back water from Kushiyara River has an effect on the water level at Moulvibazar of Manu River including that of the PRW site. The water level at Sherpur of Kushiyara River during the dry season (Jan-May) from 2007 to the present is shown in Figure 4.

Based on Figure 4, the following matters are identified:

- In April, the water levels at Sherpur are about PWD +3.0 to 4m m, except 2010 and this year. Therefore, usually there is no backwater to the PRW site in April.
- However, the water levels at Sherpur are around PWD +7 m and are rising.
- In April 2010, the water levels at Sherpur were around PWD +7m and not lowered.

(3) Recommendation
According to the information from the Meteorological Department, heavy rain in the Indian Territory is forecasted at present. Therefore, there is a possibility of no lowering water level around the PRW site. It is better to prepare the countermeasure in such case.

4. Possible Time Extension

(1) Revised Time Schedule of the PWR (as of 14/Apr/2016)
After heavy rainfall in the end of March, the construction time schedule was revised as of 14/Apr/2016, as shown in Figure 5. According to this time schedule, remaining works require about 1.5 month.
However, the water level is not lowered yet due to higher water level in the Kushiyara River, and the construction works stop until now, except the manufacturing of projected CC blocks.

(2) Suspension of the Works and Time Extension
There is an internal regulation of BWDB, which prohibits the river works from July to September, except the emergency works, In consideration of required period of the remaining works of the PRW (1.5 month) and the internal regulation, it is recommended that the suspension of the works and time extension of the contract of the PRW should be determined in the beginning of May.

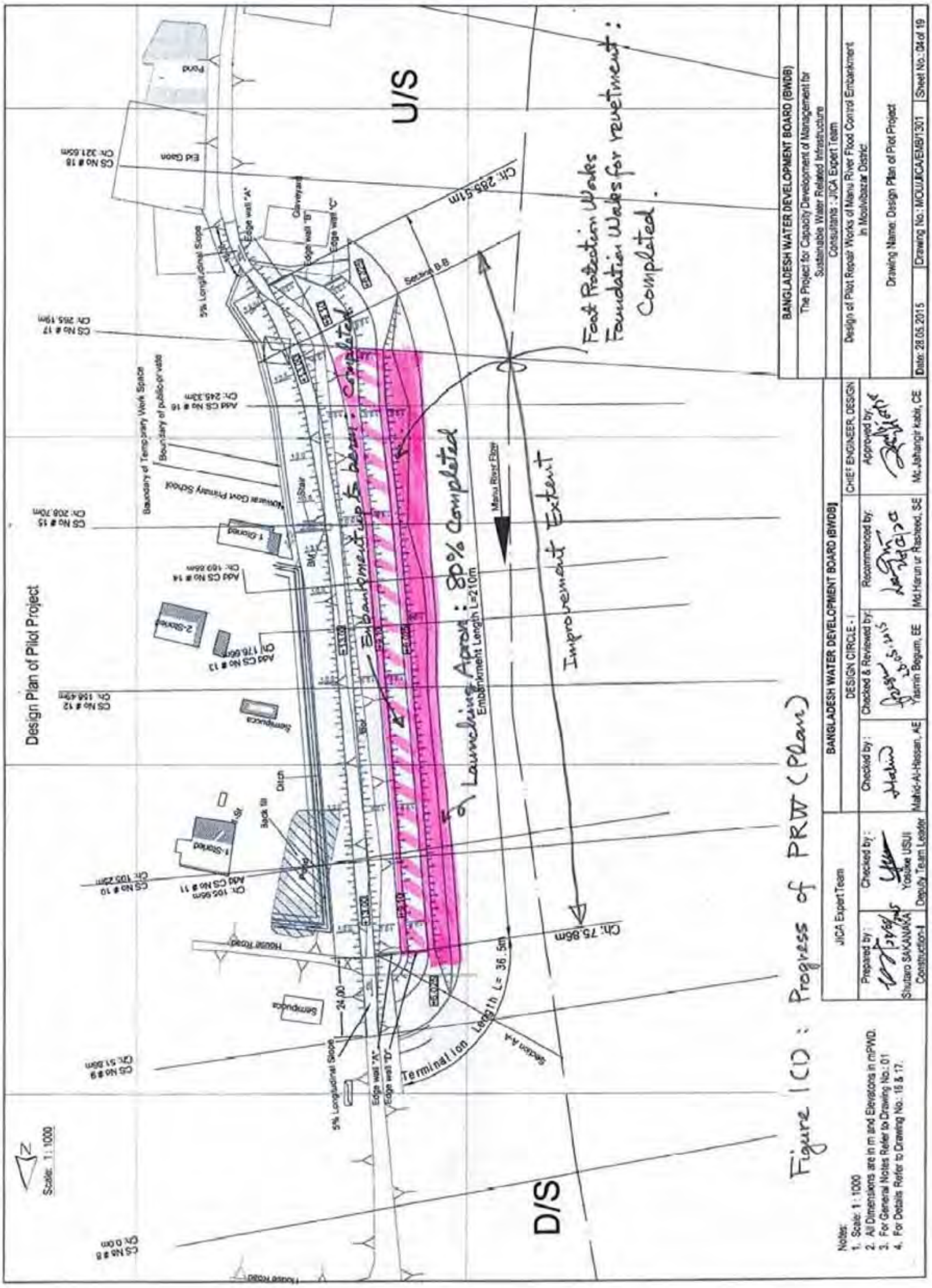
(3) Temporary prevention measures
The PRW is conducted along river side of the existing embankment and the launching apron (riverbed protection work) and the earth works up to the berm were already installed. Therefore, the present embankment is bigger and stronger than the embankment nearby. However, it is recommended to protect the embankment slope against erosion by the floods during the coming rainy season, in case of the suspension of the works.
Temporary prevention measure of the PRW was discussed in the field meeting on 19/April/2016 with the Director of Design Circle 1, and recommended as shown in Figure 6 and as follows:

- Based on the practice in the Meghna River and others of BWDB, the embankment area shall be covered with the geotextile sheet (2 mm thickness) and the geotextile sand bags (125 kg, 1 layer).
- The covering extent shall be from downstream end of transition to the existing embankment to the upstream end of transition.
- Sand gunny bags are cheaper than the geotextile sand bags. However the sand gunny bags have no durability and no weight against floods.
- Materials of the geotextile sand bags and geotextile sheet are common and easy to

Photograph: Overall View of Pilot Repair Works (from the opposite bank on 21/Apr/2016)

WL at site: PWD +7.73 m

C.C. Blocks for dumping to the Launching Apron are tentatively kept on the design berm (PWD +9.1 m) of embankment.


Design Plan of Pilot Project
Scale 1:1000
U/S
D/S
Figure 1(1): Progress of PRW (Plan)
Launching Apron: 80% Completed
Embankment up to berm: Completed
Foot Protection Works
Foundation Works for revetment: Completed.
Improvement Extent
Manu River Flow
BANGLADESH WATER DEVELOPMENT BOARD (BWDB)

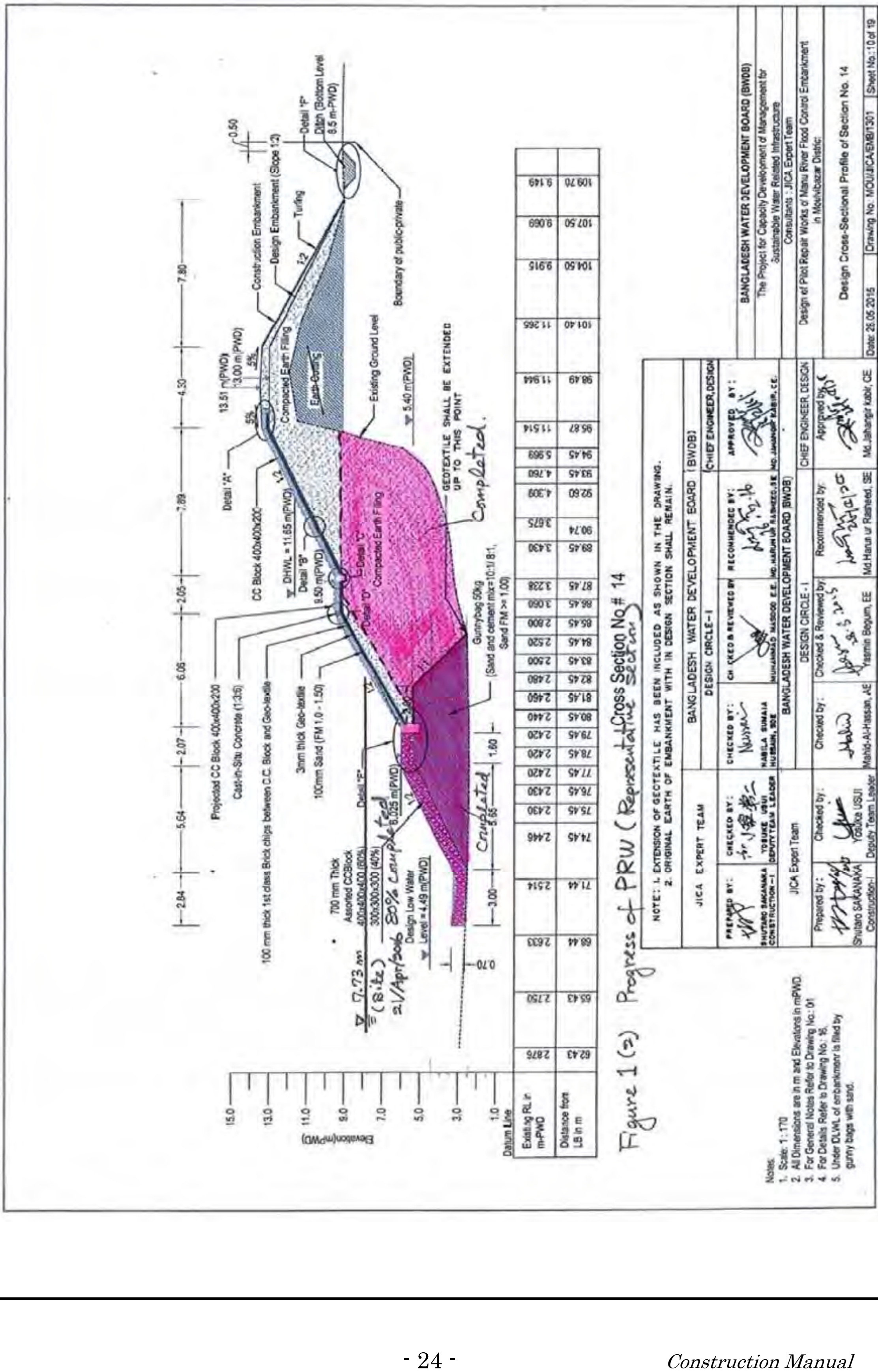


Figure 1 (3) Progress of PRW (Representative Section) Cross Section No # 14
GEOTEXTILE SHALL BE EXTENDED UP TO THIS POINT
Completed.
Design Cross-Sectional Profile of Section No. 14
BANGLADESH WATER DEVELOPMENT BOARD (BWDB)

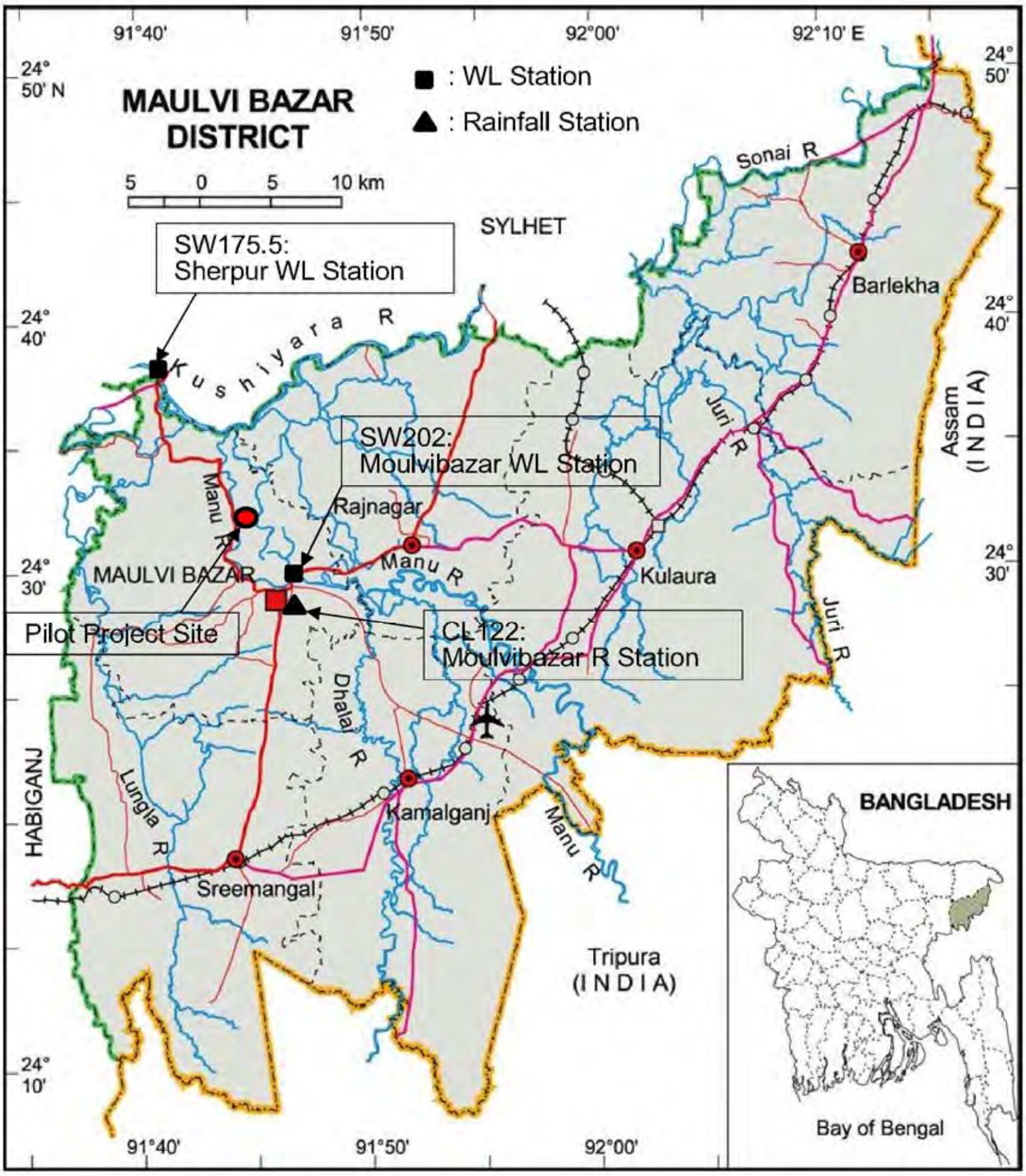


Figure 2 Location Map of Water Level and Rainfall Stations

Table 1 Summary of Monthly Rainfall and Rainy Days at Moulvibazar

Year	Jan		Feb		Mar		Apr		May	
	Rainfall (mm)	Rainy Days (days)	Rainfall (mm)	Rainy Days (days)	Rainfall (mm)	Rainy Days (days)	Rainfall (mm)	Rainy Days (days)	Rainfall (mm)	Rainy Days (days)
2007	0.0	0	52.0	5	77.0	2	294.0	11	308.0	10
2008	15.0	1	10.0	1	44.0	4	10.0	3	158.0	12
2009	0.0	0	0.0	0	14.0	1	128.0	11	340.0	13
2010	0.0	0	0.0	0	142.0	4	329.0	9	482.0	18
2011	1.0	1	0.0	0	6.0	3	45.0	7	362.0	18
2012	0.0	0	0.0	0	23.0	2	274.0	17	228.0	17
2013	0.0	0	NA	NA	4.0	1	31.0	5	572.0	21
2014	0.0	0	22.0	2	7.0	4	82.0	6	261.0	13
2015	0.0	0	13.0	1	0.0	0	347.0	11	495.0	18
2016	0.0	0	86.0	2	148.0	6	218.0	6	-	-

Note: up to 12/Apr/2016

Manu
SW202
Moulvi Bazar

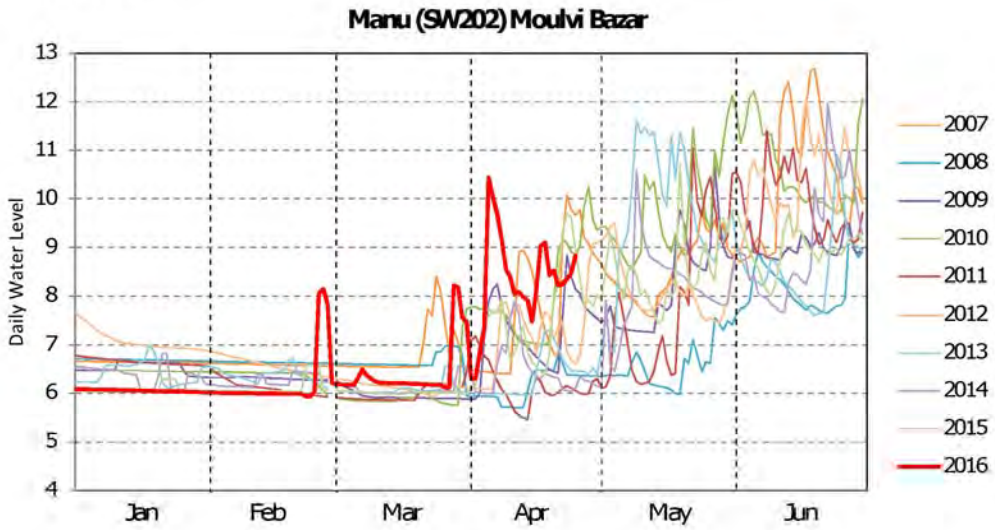


Figure 3 Water Level of Manu River (SW202, Moulvi Bazar)

Note: Moulvibazar station is located at the upstream site of the Pilot Project Site. (refer to Figure 2). WL of the Pilot Site are below about 1.5m from WL of Moulvibazar Station, in case of no backwater from Kushiyara River.

Kushiyara
SW175.5
Sherpur

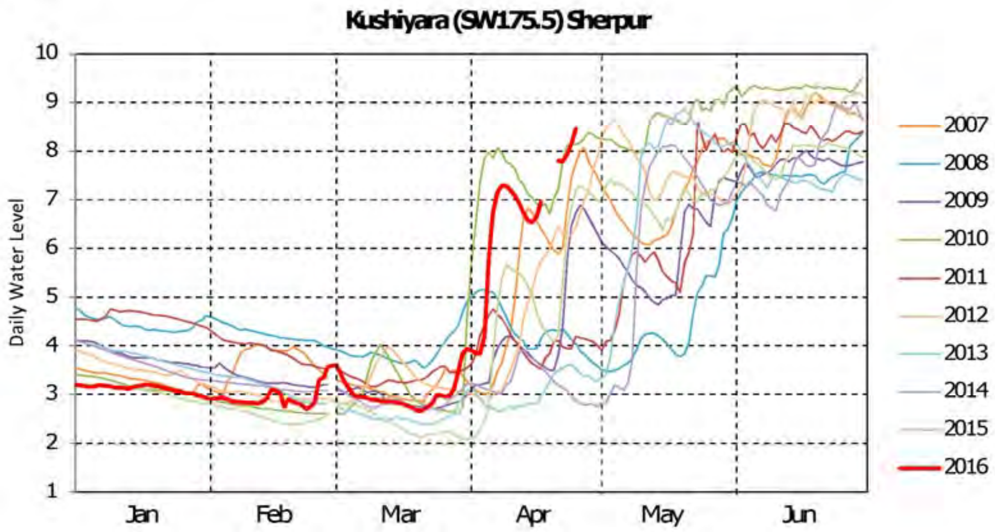


Figure 4 Water Level of Kushiyara River (SW175.5, Sherpur)

Note: Sherpur station is located at the downstream site of the Pilot Project Site. (refer to Figure 2).

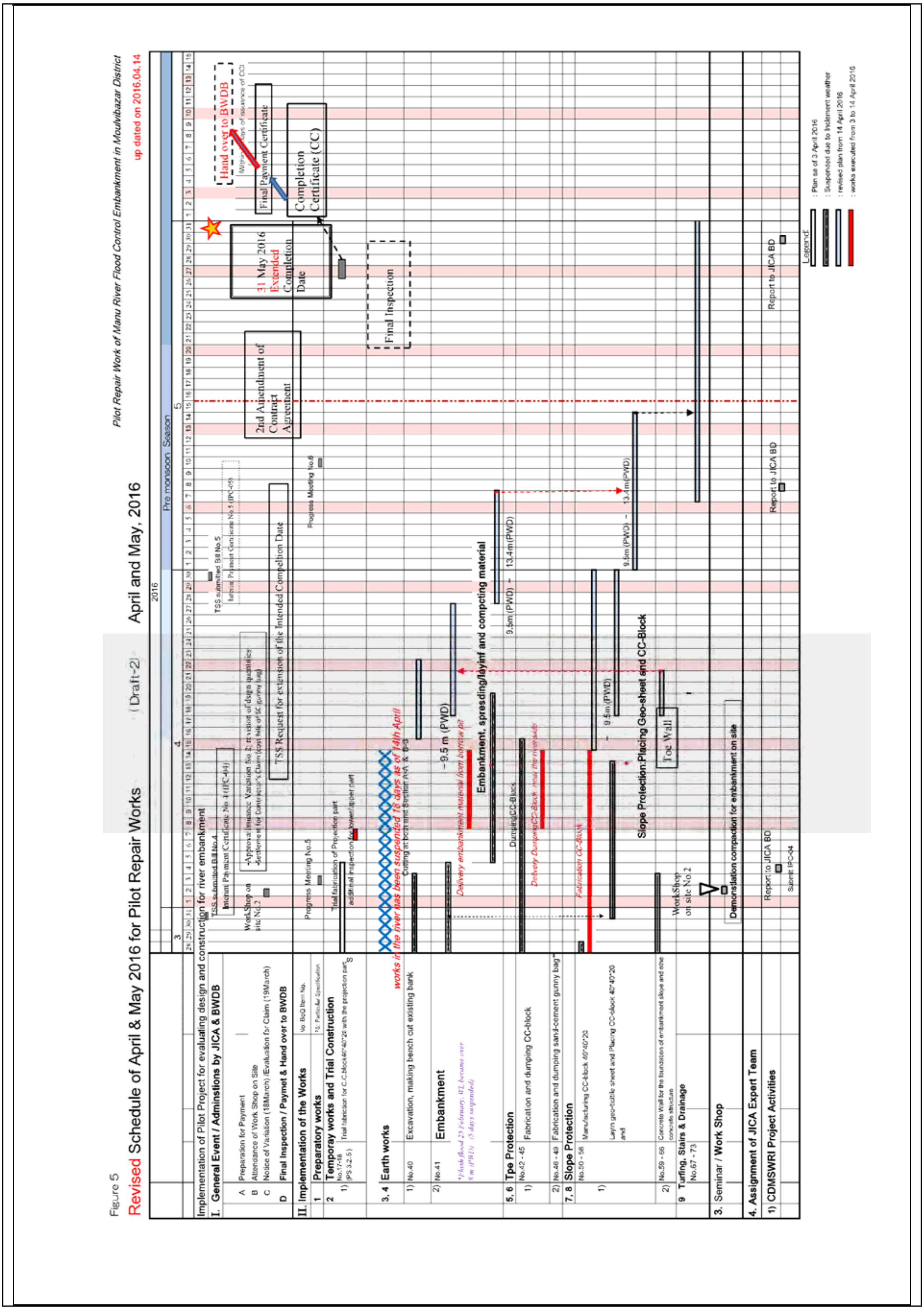


Figure 5
Revised Schedule of April & May 2016 for Pilot Repair Works

According to avobe anlaysis, including hydrogical checking, cause of delay was concludeed as follows;
1) Extra ordinaly water level and rainfall,
2) Backwater from the main discharging point (Sherpur on Kushiana River) and
3) Weatherforecast of heavy rain in the up-steam of Manu River (in the Idian teretory)

3. Detaling the modified coctruction plan/counter measurs

On the basis of the joint site inspection and disccussion among BWDB officials, JICA Expert Team and the Contractor, following measures were defined;
1) Time Extention of Intended Completion Date 15 May 2016 to 30 April 2017 and
2) Conducting a “Temporary Prevension work (TPW)” for proposed slope in the river-side (refer to Section 5.3.5 Additional prevension measures during the suspention of the Works and its example in a Pilot Works.)

4. Result and Learnings

1) Up to the end of March 2017, all Toe and Slope Protection works in the river side have been completed.
2) Unfortunately, due to unavoidable circumastances, extra ordinaly early rain fall attacked to north-east regions of Bangladesh including Moulvibazar site and other reasons, the Work had to be extended to another few weeks. Actual completion date was the latter half of May 2017.
3) Importance of planning shedule
When procuring entity consider and design the construction perio for the project, past years hydrogical data at the proposed construction site, including water level and rain fall, should be studied carefully.
4) Plural seasons of construction period
If the work volume/scale is relatively large and/or expection capability of the work-force is limited and the due to site condition, 2 years or more years construction period is to be recommended . For instant first year, the Works includs only for Toe Protection works and earth work/embankment works, and secondary year the Slope Protection works are carried out.

Section 1.4 Supervision of Construction

1.4.1 Objective for Supervision

The objective of construction management is to systematically and sufficiently make organization functions, execute safe and economical construction, and smoothly ensure the planned quality and construction period.

Furthermore, the rationalization of construction management is an essential condition and more practical management is required. Without this sort of management, engineering innovations will fall short of their potential. The main contents of such construction management are shown in Table 1.4.1.

Table 1.4.1 Construction work management

Construction management	Progress management	Compare the plan and actual schedule
	Quality control	Compare the design and construction quality
	Completion management	Compare the design and actual form dimensions
Machinery management	Operation management	Improve machinery operation rate and work efficiency
	Maintenance	Maintain machinery
Safety management	On-site management	Safety measures directly related to site
	Accident prevention	Prevention of accidents related to workers and third parties
	Flood protection measures	Prevention of water damage from disasters
Environmental protection	Various countermeasures for environmental impact	

1.4.2 Quality Control

1) General

Faults found during inspections are usually not an easy matter to be repaired and, even if possible to repair such faults will cost a considerable amount of labor, time, and money. Quality should therefore be rationally managed during construction and work must progress so that construction definitely passes inspection.

Objective of quality control

Quality control is a method for economical construction that sufficiently satisfies the quality standards described in the design and specifications. The objective is to prevent construction faults before they occur, reduce variations in quality, and make the construction work more reliable.

Embankment quality control

Most of the quality control in earthwork usually deals with embankment materials and degree of compaction, and also sometimes with base material.

As soil is the material used in embankments, it is difficult to ensure homogeneity. It is therefore vital to check quality daily to obtain homogeneity.

To manage embankment compaction, a density test using the sand replacement method is generally employed as the quality control method to measure compacted soil density and water content. And radioisotope (RI) gauge method might also be conducted for such test.

2) Quality control procedures

Quality control can generally be implemented based on the following policies and it is shown on Figure 3.3.1;

(1) Determining quality characteristics

Quality characteristics such as grading of fill material and compaction degree of embankment body should be determined as control items for each work process in the river embankment construction.

In contract work, specifications usually show the quality characteristics to be managed in advance and the quality is managed with a quality control test that corresponds to the characteristics.

(2) Determining work standards to ensure quality standards

The methods and order of work must be decided to ensure quality standards, so it is necessary to determine work standards with consideration for previous experience and construction capabilities based on the design and specifications to have sufficient allowance in the schedule.

For embankment, work standards are to determine methods that are appropriate for each type of soil and standards, including compaction methods, types of machinery and weight to be used for compaction, thickness of spread soil, construction water content and number of compaction passes. These standard construction methods should also be determined in principle through ***trial construction***.

(3) Checking quality

When quality does not meet the standard values or works are not consistently carried out according to the daily management results, reconstruction such as re-compaction or review the standard work procedures are to be made.

1.4.3 Measurement of works

Civil works are often large in scale, and it is extremely difficult to do rework, even if defects are found after completion. Thus, it is important to check that construction meets the standards and dimensions indicated in the design drawings and specifications during construction work—a process called “measurement of works” control.

Finished forms in earthworks include river embankment alignment, width of crest, slope width and length, gradient, and standard elevation.

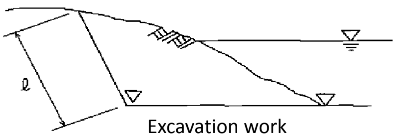
Tolerances are the limitation values of the differences between design values and the completed work. Table 1.4.2 shows examples of standard management values or tolerance.

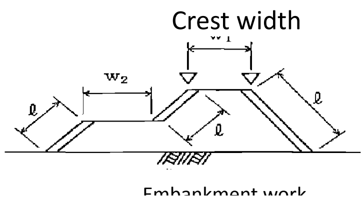
Table 1.4.2 Example of standard management values for measurement of works

(mm)
Table 5.3.3 Examples of work progress control standard values

		Measurement Items		Stand. Value or tolerance	Measurement Standards
River earthwork	Excavation work	Reference elevation		±50	One place for work lengths of 40 m (50 m if measurement spacing is 25 m), two places for every work spot for lengths under 40 m long (or 50 m). Reference elevation measured at both ends of excavated area.
		Slope length L	L＜5m	－200	
			L≧5m	Slope length – 4%	
	Embankment work	Reference elevation		－50	One place for work lengths of 40 m (50 m if measurement spacing is 25 m), two places for every work spot for lengths under 40 m long (or 50 m). Reference elevation measured at the shoulder of each slope.
		Slope length L	L＜5m	－100	
			L≧5m	Slope length – 2%	
		Width w1, w2		－100	

Note: Payment shall be made by work-done quantity





(quoted by “ River Embankment manual”)

Chapter 2 Preparation & Temporary Works (200 Site Preparation)

Preparation work involves making arrangements for the main construction. Whether it is done properly or not has a huge effect on the efficiency, quality, economic efficiency and schedule of the construction. It is an extremely important part of proper construction management. It is important to consider preparation work carefully in the construction period, work scale, site conditions and other project details, selecting the methods best suited to working conditions in order to make work more efficient.

Section 2.1 Surveys & Profile Stake & Finished profile stake

2.1.1 Preparatory surveying

Preparatory surveying before the work is important to fully understand how the design drawings relate to the site. If preparatory surveying reveals inconsistencies between the design drawings and the site, it is important to promptly survey the causes of those inconsistencies and take proper actions.

(1) Setting temporary Bench Mark

Temporary bench markers are to be set up using official Bench Mark (BM) on the foundations of permanent structures and other immovable objects. When such objects are not usable due to site circumstances, temporary bench markers are to be installed. They are to be set outside of construction areas on solid ground where there is no chance that the pillars will be damaged by general traffic, and they require proper protection.

Elevation for temporary bench markers is to be determined by horizontal leveling from existing official BM and confirming positions by referencing at least one other existing official BM. Temporary bench markers cannot be placed unless this confirmation is done. The elevation of the temporary bench markers must be verified regularly.

(2) Placing backup pillars

The alignment markers, transverse markers and backup pillars needed for the work must be provided. Backup pillars must be put in places where the original position of alignment markers and transverse markers will be known if excavation or embanking changes the topography.

Note that backup pillars will preferably be put somewhere off site with hard ground where there is no risk of losing the pile. If the pile has to be put within the site, a suitably protected location must be selected which will not obstruct operation of work machinery or the loading and unloading of materials.

(3) Checking and setting boundary markers and survey markers

Boundary markers and survey markers must be checked before construction.

Boundary markers may be broken, removed or become lost if any time has passed between work site acquisition and tree clearing before construction starts, resulting in discrepancies. Thus, markers need to be verified to make sure that they are in the correct positions.

In addition, the concerned authorities must be consulted and proper measures taken before repositioning existing distance markers or bench markers when construction requires them to be repositioned.

(4) Introduction of survey instruments

Followings are introduction of some main instruments for survey works in river embankment works.

a) Auto-Level

To survey the elevation of the ground and the structure, “Level” is essential and basic instrument. Now “Auto Level” is broadly used in construction field. Left is one latest model.

<SOKKIA:B20/B30A>

Ph. The picture was taken for Cross Section Survey by TSS (JV) carried out at field on 13 &14 December 2016, with checking by JET SV Team as Pilot Repair Works in Manu River

b) Total station

To topographic survey, "Total Station" is now broadly used. Previously Theodolite instrument have been used for such survey but "Total Station" has more convenient function, such as measure the distance and calculation/sophisticated applications.

Below is one of example of latest model by "SOKKIA".

c) UAV (drone)

Last only few years, newly producing UAV (Unmanned Aerial Vehicle, or "Drone") has been developing on many activities. Also in construction field, it will be very useful instrument, especially survey works, including river embankment works, and emergency information collection on the site.

Ph. Lying UAV

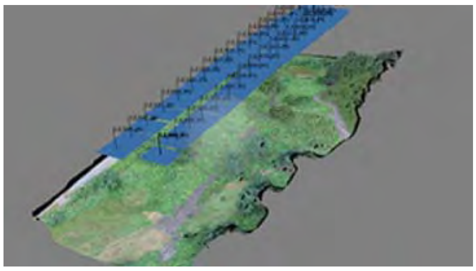


Fig. 3D mapping from UAV taken photographs

Left Ph. Operating UAV on the river flood plain during the flight to take photos in order to make topography drawing for the damaged river banks

2.1.2 Finished profile stake

Finished profile stake, or grade stake / leading frame, serve as references in the course of the target work and must be firmly installed to remain in place during construction. They must be inspected regularly and verified and corrected when doubts exist.

As a rule, finished profile stake will be placed at 10-meter intervals on straight reaches of the river and at five-meter intervals on curved and other complicated reaches, but it is best to place them at closer intervals when necessary.

Note here that there are finished profile stake for cutting, embanking, rubble work and other purposes. The type of finished profile stake to use must be selected to fit site conditions and the target work.

Section 2.2 Temporary Access Road

2.2.1 Investigation for Temporary Access Road

Used in the transport of equipment and materials as well as soil and sand, access road for construction is vital to the success of the project. As such, the existing road network close by to the work site in particular is crucial and must be sufficiently surveyed before the work.

When road is to be newly constructed or improved, there are a few points which must be kept in mind in the survey. It is required to check the terrain and the geological features on the route and it is need to determine the route and road structure rationally with setting the objective of usage and volume of traffic considering for local land use plans after the embankment body is complete.

2.2.2 Temporary works for Access Road

There are two kind of temporary access road; one is used for the existing public road and the other is the use newly constructed temporarily.

Basically the existing public road is to be used as access road, but in case it is difficult to use the public road newly construction or improvement of the road will be needed.

1) Using existing public Road

Firstly check the status of traffic condition and secondary assess the proposed work load during the usage for access road. And if necessary, the road will be strengthened. Discussion will be made with the parties concerned the road, Representative of maintenance department of RHD and others, regarding the repairing works for that road or traffic restriction and other issues of the road.

2) In case of newly construction for a temporary access road

As a temporary works of access for construction material and equipment, the temporary road will be made as possible as small in scale and structure. In case the access road is newly constructed, it must be considered to minimize the environmental impact to surroundings as possible as little. Basically, the temporary works shall be restored as well as before.

Section 2.3 Temporary Construction Yards

It is necessary to provide fabrication yards for CC-block manufacturing, temporary stock yards for construction material such as geo-bags, temporary stockpile area for imported embankment material and temporary construction area for temporary office and storage and vehicles parking space during the construction nearby the site.

In the river embankment works, these temporary construction yards will be placed basically at the country side because of water rise at flood.

Where the existing farmland is utilized for the temporary yards as lease, basically such farmland shall be restored for cultivation after the completion of the Works.

Chapter 3 Earth Works (300 Earth Works)

Section 3.1 General items of Earth Works

3.1.1 Soil Movement Plan

The plan for allocating soil to be excavated and embanked is called the soil movement plan. As this plan will greatly affect work arrangements, selection of construction machinery and the schedule, the soil movement plan must be prepared to allow soil to be delivered economically.

1) Deformation of soil volume change rates

When excavating, transporting and constructing the river embankment with soil, the volume of soil will differ based on whether the soil is in the ground, loosened from the ground, or compacted. Accordingly, soil volumes at each state (or location) must be determined in advance in order to formulate the soil movement plan.

The three soil states and their relations with earthwork are as follows:

Ground soil volume	Volume of soil to be excavated (soil in the ground)
Loosened soil volume	Volume of excavated soil to be transported (excavated, loosened soil)
Compacted soil volume	Volume of compacted soil in embankment (compacted)

The soil volume ratios for other states against the ground soil volume, are called the soil volume change rates and are defined as follows:

$$L = \frac{\text{Loosened soil volume } (m^3)}{\text{Ground soil volume } (m^3)} \qquad C = \frac{\text{Compacted soil volume } (m^3)}{\text{Ground soil volume } (m^3)}$$

The state of soil differs according to its usage and keep it mind that each volume of soil of each phase shall be different.

Thus, each volume of soil shall be calculated according to its usage when planning soil allocation.

(a) For deciding the necessary volume of embankment;

Volume of embankment = (Ground soil volume) × C

(b) For deciding the necessary volume of transportation;

Volume of transportation = (Volume of embankment) ÷ C × L

Soil volume change rates can be determined either through simple measurements, trial construction or through estimates based on results of past work. Table 3.3.1 gives standard ranges for soil volume change rates by soil class.

Table 3.3.1 Soil volume change rates

Class Name		Change Rate L		Change Rate C	
Key Classes		Reference Value	Normal Range	Reference Vale	Normal Range
Gravelly soil	Gravel	1.15 (1.20)	1.05~1.25	0.95 (0.95)	0.85~1.05
	Gravelly soil	1.20 (1.20)	1.10~1.30	0.95 (0.90)	0.85~1.05
Sandy soil and sand	Sand	1.2 (1.20)	1.10~1.30	0.95 (0.95)	0.85~1.05
	Sandy Soil (normal soil)	1.20 (1.20)	1.10~1.30	0.90 (0.90)	0.80~1.00
Cohesive soil	Cohesive soil	1.30 (1.30)	1.20~1.40	0.90 (0.90)	0.80~1.00
	High water content cohesive soil	1.25 (1.25)	1.15~1.35	0.90 (0.90)	0.80~1.00

Notes:

1) Maximum values are standard values closer to consolidated or packed soil values, while minimum values are closer to loose soil values.
2) Standard values within parentheses are from the Cost Estimate Standards for Civil Work from the Japanese Ministry of Land, Infrastructure and Transport (2008 Ed.).

2) Preparing the soil movement plan

Soil allocation is a central facet of the construction plan. Normally, river embankment use soil excavated from the setback area or other sources as fill material. However, in some cases where soil is not fit for fill material, or long transport distances make it an economic burden. In such cases a suitable soil movement plan must be considered, outlining location of excavated soil to be embanked, rules for dumping of purchased soil, and other details.

The basic concept is to plan soil movement to minimize “the volume of transported soil-by- transport distance”, and also considering for the following:

a) Make a plan that will allow construction to go smoothly, giving sufficient thought to coordination between the earthwork plan and the schedule for any sluiceways, sluice gates and other structures.
b) If excavated soil is used for embankments, soil type and quality may differ greatly. In such cases, the plan should be effectively made for different soil by type and quality to complete the embankment bodies as effectively as possible.
c) For transport facilities, consider building a temporary bridge if excavation area and the embankment are on opposite sides of the river, or building a temporary road if they are

upstream or downstream from each other. And also take consideration for ship or barge transportation in such cases.

3.1.2 Construction Machinery Selection

When river eathwork is carried out mainly by consrtuction machines, selection of construction machinery has large effect to the construction.

Which kind of construction machinery should be selected with consideration of the overall work scale, soil volume, soil quality, terrain, construction period and other working conditions, but also of economical factor and familiarity of machine operators

1) Applicable equipment by type of work

Table 3.1.2 gives the construction machinery suited to earthwork as classified into excavation, loading, transport, spreading, compaction and other tasks. Some is used on a single type of work, while others can be used in succession on several type of work.

Table 3.1.2 Applicable machinery by type of work

Type of work	Type of construction machinery
Excavation	Shovel excavators (power shovels, backhoes, draglines, clamshell buckets), bulldozers
Loading	Shovel excavators (power shovels, backhoes, draglines, clamshell buckets)
Excavation/ loading	Shovel excavators (power shovels, backhoes, draglines, clamshell buckets), crawler loaders, wheel loaders
Excavation/transport	Bulldozers, scrape dozers, scrapers
Transport	Bulldozers, dump trucks, belt conveyors
Spreading	Bulldozers, motor graders
Compaction	Bulldozers, tire rollers, rammers, tampers, vibrating compactors, vibrating rollers, road rollers

The construction machinery used may differ for one type of work if site conditions are different. As such, it is important to fully consider the trafficability and soil conditions, work site size and construction scale, construction period, familiarity with the machinery and other conditions when selecting a type or standard of construction machinery.

2) Models applicable for different soil conditions

For earthwork for river embankment, construction machinery selection will need to account for soil conditions in the following cases:

a) Soil on the site is soft, and trafficability is an issue
b) Method or model is being selected when compacting
c) Excavation method is an issue due to soft rock or extremely densely packed soil

(1) Trafficability

When operating construction machinery on soft soil, work efficiency will depend greatly upon the soil type and water content. In cohesive soil with particularly high water content, construction machinery will create ruts which weaken the soil, sometimes to the point that operation is impossible.

(2) Adaptability of compactors

The machinery to be used for compaction is to be selected based on construction conditions, such as soil quality of fill material, type and scale of construction, and the characteristics of compactors.

It should be noted that bulldozers are the standard compactor for river embankment. Also, bulldozers are highly versatile, economical machines that can do the entire series of banking work in one machine, from spreading to compacting.

3) Notes on machinery selection

It must be noted that items such as construction period, work scale and machine familiarity are all important elements in selecting the machinery. Also, transport is a key task in earthwork that accounts for a large percentage of earthwork costs. As such, selecting suitable machinery for transport is essential in moving and transporting large volumes of soil.

(1) Construction period, work scale and familiarity with the construction machinery

Often in earthwork for river embankment, large volumes of earthwork must be accomplished in a short period. In general, construction machinery is to be selected to fit the scale of the work. Large machinery is generally ideal for use in large-scale work, and likewise small machinery is ideal for use in small-scale work.

(2) Transport distance and work site area

As soil transport is a key task in earthwork and transport expenses are a large percentage of earthwork costs, it is important in work where large volumes of soil are being moved to select the right transport machines. Pay particular attention to details such as transport distance, gradients and work site area when selecting transport machines.

While the machinery suited to the transport distance must be selected on a per-site basis, Table 3.1.3 gives the standard transport distance ranges for each transport machine. In general, bulldozers are often used for transporting short distances, and dump trucks are used for transporting longer distances.

Table 3.1.3 Soil transport distances for transport machinery

Type of machinery operated	Transport distance
Manual	Under 40 m
Bulldozer	Under 60 m
Scrape bulldozer (150HP)	40 ~ 250 m
Towed scraper	60 ~ 400 m
Self-propelled scraper	200 ~ 1,200 m
Shovel excavator, Tractor shovel + Dump truck	Over 100 m

Notes:

2) In special cases, tractor shovels are suited for use in excavating and transporting under 100 meters.
3) For transport distances of 60 ~ 100 meters, compare use of a bulldozer, dump truck and other machines according to the site conditions.

(Quoted from "River Embankment manual")

Also note that machine selection must account for how much area there is at the excavating and loading points on the work site. Scrapers in particular require wide areas to turn. Also, width of the transport route must be confirmed to select the size of transport machinery.

Section 3.2 Excavation

3.2.1 General

Excavation methods will differ by the soil excavated, terrain and other conditions.

The methods for excavation and transport in river earthwork should be selected based on whether excavation is just to expand the river channel or the excavated soil is planned for use in embanking or other work. If excavated soil is to be used, methods must account for items such as balancing soil volume with the soil movement plan and controlling fill material quality.

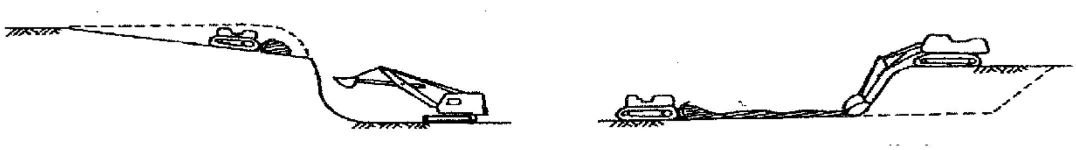
3.2.2 Excavation for soil

(a) Mechanical excavation

Where machines are used for excavation for soil, shovel excavators, bulldozers, tractor shovels and similar machinery are used.

If dump trucks are to be used as the main machine of soil transport, shovel excavators will be used to both excavate and load the soil. Note that often bulldozers can be effectively combined with such shovel excavators as a support excavator for piling soil and finishing.

Also, it will generally be helpful for the excavator to work from an elevated position in river earthwork when water flowing from small leaks can impact construction. This is an important point to consider when selecting your combination of machinery.



a) Excavation height exceeds shovel range　　　　b) Piling with a bulldozer

Figure 3.2.1 Combination with supporting excavator

(quoted by " River Embankment manual")

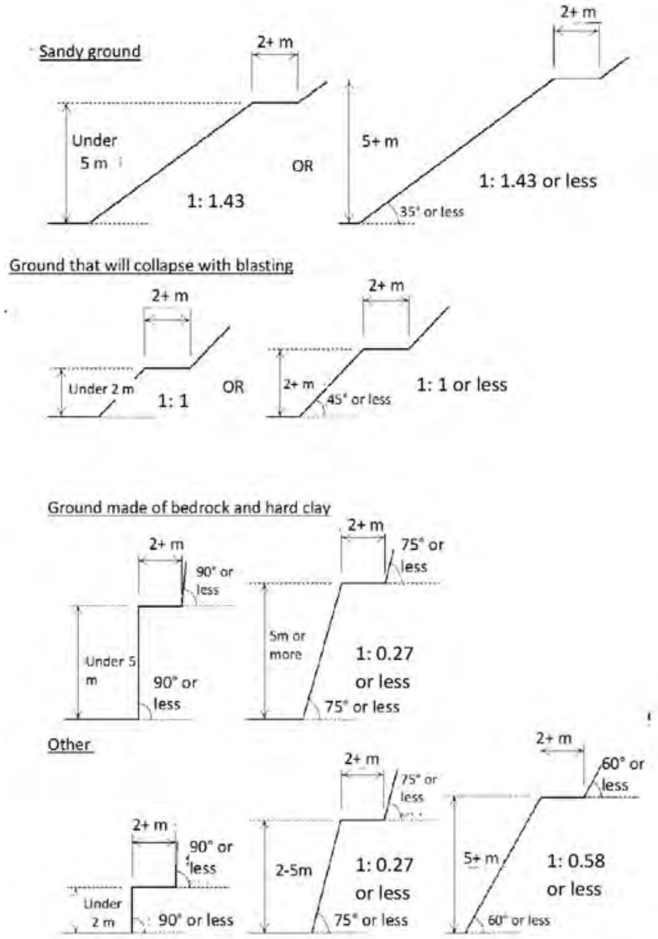
(b) Manual excavation

Manual excavation is used when mechanical excavation would be uneconomical, or in smaller work where machinery cannot excavate or move freely in certain areas, such as mechanical excavation close to the finished surface or with structures nearby due to the plan. Finished surface excavation, meaning excavation of the last 5 to 10 centimeters left after mechanical excavation, is generally performed manually. An effective method is to devise a way to leave extra soil alongside mechanical excavation, finish the surface manually and deliver out the excess soil by machine.

With works digging below the ground surface in manual excavation, safety is paramount.

The lesser gradient of the slope is more safety than the steeper gradient of which. Usually the gradient are described the ratio of horizontal 1 to vertical x, such as 1:1, 1:1.5, 1:2 or 1:3. And there are also some "gradient standards for excavation in manual" based on rules of thumb determined by depth of excavation and ground type.

Left figures is one of introductive example for such standards.



Reference Figure 3.2.1: Experience-based samples of gradient and height for manual excavation

(source; "Occupational Health and Safety Regulations" Articles 356 and 357, Japanese)

3.2.3 Notes on Excavation works

(a) Excavation must be performed properly according to the design section without disturbing the original ground. The finished surface has to perform its designed functions of directing river flow, and excessive dig depths or an uneven surface will disrupt normal flow. Thus, great care must be taken when close to the finished surface. Tolerances for unevenness on the finished surface are generally specified as (+) or (-) 10cm, and care must be taken to keep intervals to a minimum for the finished profile stake showing excavation height during the work.

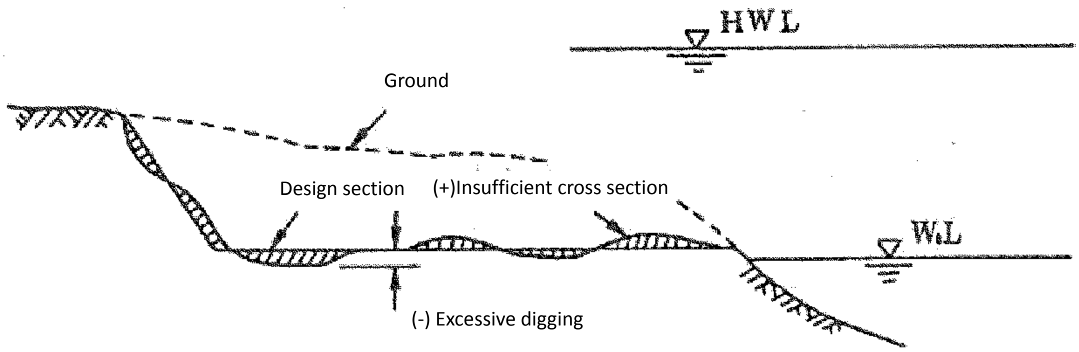


Figure 3.2.2: Example of section irregularities

(Figures 3.2.2, 3.2.3 & 3.2.4 are quoted by "River Embankment manual")

(b) Excavation methods for river channels, and low flow channels in particular, must not significantly disrupt the direction of flow during the work.

i. In general, excavation of a section series is to proceed from downstream to upstream. As shown in Figure 3.2.3, starting excavation upstream will change the direction of flow on the excavation surface, causing turbulence. This can not only cause deep holes and scouring on the riverbank or river bank where the water hits, but can also scour the soil from the area being excavated.

ii. If a wide area across the river is to be excavated as in Figure 3.2.4, excavation should be divided into several blocks almost parallel in flow, starting from the embankments and moving upstream.

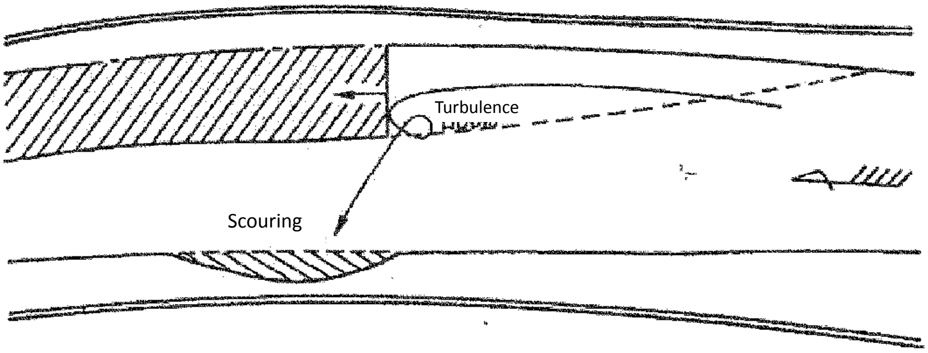


Figure 3.2.3: Example of incorrect excavation order

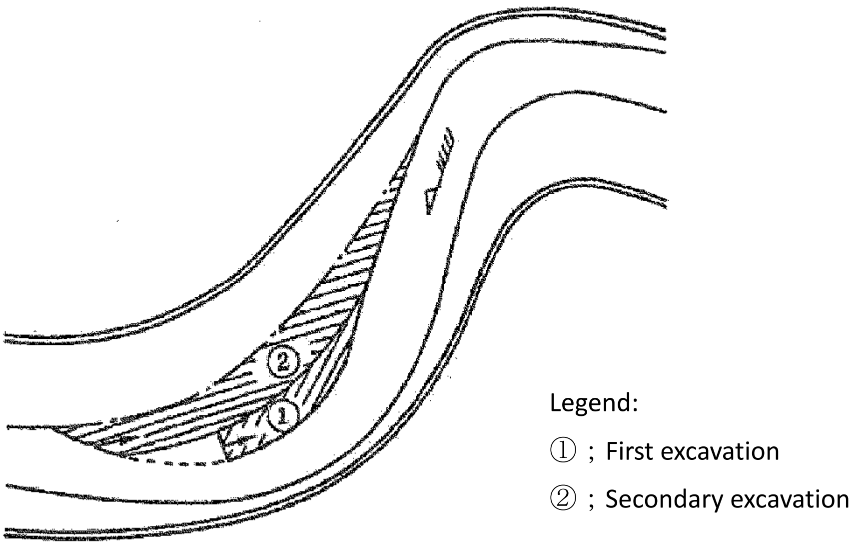


Figure 3.2.4: Example of correct excavation order

(c) Measurement for high water content

Excavation methods must be fully considered based on how excavated soil is to be used. If the soil is to be used in embanking work, water content is of particular concern. The soil must be transported directly to the embankment and compacted to specified levels. In excavation of setbacks and low flow channels, water content should thus be lowered to be as close to optimal as possible. Water content can be lowered with cut-and-cover or cofferdams to lower groundwater and river water levels, installing ditches for pumping out or temporary staging of the excavated soil. With cohesive soil, as lowering groundwater levels can prove difficult, methods such as dividing excavation area relatively into thin layers in the hopes of drying from the ground surface must be considered.

(d) Shelter for machinery

A plan is needed for swiftly evacuating excavation machinery to a safe location in case of flooding. Given that excavating machinery generally moves slowly and water levels will rise quickly in short time after rainfall, especially on smaller rivers, a shelter location that can be safely reached in time must be prepared in advance.

(e) Removing old river embankments

When removing old river embankments, work should proceed from downstream to upstream, from the country side to the river side in order to avoid disrupting river flow direction in a flood.

Section 3.3 Management of embankment material & Transportation

3.3.1 Borrow pit and dumping area surveys

1) Borrow pit and dumping area surveys

It is advisable to be somewhat selective about fill material. River embankments require firm fill material to be sufficiently safe against seepage water during flooding. A separate borrow pit will be necessary if the soil excavated on site is not suitable as fill material, or there is not enough fill material. In selecting a borrow pit in such circumstances, the borrow pit that is most suitable and economical for that work must be selected. First, several candidate sites are to be selected based on the quality and volume of soil that can be borrowed, with each candidate surveyed for various conditions and sufficiently discussed. Survey conditions shall include terrain, soil quality, transport distance, transport route, access roads used, surrounding environment, buried cultural assets and local regulations.

Soil quality for the borrow pit will affect by factors such as embankment quality, trafficability of construction machinery and compaction methods, meaning that investigation will be needed to account for things like soil consistency, grain size and natural water content.

In earthwork construction, the set back is often used as the borrow pit. The water content of the fill material should be checked in particular, taking care that it will not hinder compaction. Adjusting of water content or other measures may be advisable in some cases. If it is necessary, stock pile area should be provided

If soil conditions are roughly similar, the borrow pit with the shorter transport distance will generally be economical, but it is also important to compare transport routes in addition to distance. In particular, borrow pits connected by routes with residential areas, school routes, highly trafficked sections, railroad crossings or traffic signals should be avoided if possible. If the borrow pit will require establishing a new road off of existing roads for construction traffic, or if part of the road will need paving or reinforcement due to the amount of construction vehicle traffic, such work will greatly impact construction costs. Such factors must be fully considered in the survey.

2) Survey of loading/unloading sites for soil generated by construction

In recent years, land shortages near big cities, flood controls and environmental issues have made it increasingly difficult to secure dumping areas. Upon sufficiently studying the work periods, soil conditions and other factors on the soil loading and unloading sites, consider how the soil generated by construction can be used appropriately. In addition to making effective use of as much soil as possible to the extent that these site factors are met, stockyard use and other things also need to be considered.

3.3.2 Transportation

Whether transport should be performed by construction machinery or manual labor is to be decided through comprehensive consideration of the work scale, work efficiency and scheduling, site conditions, and economic efficiency compared to the price of the work.

(1) Transporting soil with bulldozers

Considering general construction machinery, bulldozers can increase excavation and transport capacities over short transport distances up to 70 meters. As such, bulldozers see a wide variety of uses from excavation to dozing, and covering large excavation, clearing and access road creation in small-scale work.

As site conditions allow, excavating and dozing on a downgrade will maximize pulling power and blade capacity for better efficiency. As work scopes are normally not uniform and change often, safety precautions are a must, especially on sloped land.

(2) Transporting soil with dump trucks

Dump trucks are more economical than other methods the longer the transport distance, but they also see frequent use over relatively short distances. Poor transport routes will greatly impact efficiency, both slowing speeds and limiting load capacity.

3.3.3 Managing fill material

Ideal fill material will be easy to spread and compact, strong after compaction, have low compressibility, be robust against erosion from river water and rain, and not expand much when absorbing water. If treated properly, even cohesive soil with high water content can be used as material in the embankment body with satisfactory structural stability. Except for particularly poor material, such soil does no longer need to be seen as unsuitable as fill material in most cases.

The following action is generally taken for soil that is rated as poor embankment body material or for which trafficability cannot be guaranteed:

a) Mix soils with different characteristics (size control)
b) Dry out soil to reduce water content
c) Stabilize the soil with additives

(1) Size control by mixing soils with different characteristics

As the main purpose of this soil is for river embankments, this method is used to lower permeability by mixing small particles into sandy soil with high permeability, or to prevent cracks from drying by mixing coarse-grained soil into cohesive soil.

(2) Drying soil to adjust water content

The goal of water content adjustments is to match the water content of embankment body materials with the water content ranges stipulated in compaction regulations. Available methods include aeration for drying, trench excavation to reduce water content and spraying water.

(a) Aeration

Aeration is drying to lower water content through spreading, leaving, and mixing prior to compaction.

Aeration plan should be made with consideration that aeration work requires a wide area and is efficient in the dry season.

(b) Trench excavation

This involves excavating a trench underneath the cutting work area prior to excavating a borrow pit to decrease the water content of the material and lower the ground water level. Lowering the natural ground water content by trench excavation will also increase work efficiency.

(c) Spraying water

As spraying water increases the water content of materials, water is sprayed during compaction after soil spreading. The necessary water content can easily be calculated as the difference between the natural water content of materials after spraying and the necessary water content for compaction.

It is important to make the soil adequate to the construction water content for compaction in order to increase the degree of compaction and to prevent from water adsorption with spraying water in case clay or sand is dry.

(3) Stabilizing the soil with additives

Sometimes soil stabilization treatment is used to improve trafficability and strength deficiencies. This method uses lime and cement as additives, stirring the additives into the soil with mixing machines such as backhoes and stabilizers. As this soil stabilizing treatment methods will increase the cost of embankment construction, however their effect and economic efficiency should be considered carefully.

3.3.4 Quality control of fill material

Perform a soil compaction test in advance on fill materials to understand the maximum dry density (MDD) and optimum water content (Wopt) to determine the quality control standard. Also perform the following tests as necessary to determine appropriateness as fill materials:

- Soil particle test
- Soil particle density test
- Soil water content test
- Soil liquid limit and plastic limit test
- Soil unconfined compression test
- Soil triaxial shear (compression) test
- Soil consolidation test
- Soil shear test
- Soil permeability test

If the results of these tests show different fill material characteristics compared to the design, the following measures are generally enacted:

a) Size control by mixing soils with different characteristics
b) Adjust water content if it is dry or moist
c) Stabilize the soil with additives

Items to be investigated for quality control of fill materials are described in Table 3.3.1.

(1) Size control by mixing soils with different characteristics

Carry out the work in accordance with “3.3.3 (1) Size control by mixing soils with different characteristics.” During this work, perform the following quality control tests for each material and mixed materials to confirm that the quality of each meets the prerequisites:

- Particle size distribution and cumulative particle size distribution tests
- Compaction test
- Penetration test

(2) Adjust water content if it is dry or moist

Management of water content is essential for embankment material quality control to meet the prescribed degree of compaction. On-site water content adjustment methods should follow “3.3.3 (2) Drying soil to adjust water content.”

Measure the water content before and after adjustment to check that the quality improved.

(3) Stabilizing the soil with additives

Stabilizing the soil with additives—mixing cement or lime into embankment materials—improves soil, and a mix proportion test should be performed to determine the appropriate amount of additives.

Lime and cement can be used as additives and stabilizers or other machinery can be used for mixing. When using soil stabilization for the entire embankment, the methods must be thoroughly reviewed for effectiveness, impact to the environment, and economic efficiency.

Table 3.3.1 Quality control test and frequency (examples)

<table>
<tr><th>Class</th><th>Test category</th><th>Test Items</th><th>Test Method</th><th>Testing Standards
(frequency or criteria)</th><th>Notes</th></tr>
<tr><td rowspan="3">Materials</td><td>Neces-sary</td><td>Soil compaction test</td><td>ASTMD1557</td><td>Initially and when soil quality changes</td><td>・Maximum dry density (MDD) and optimum moisture content (Wopt)</td></tr>
<tr><td rowspan="2">Other</td><td>Soil particle test
Soil particle density test
Soil water content test
Soil liquid limit test
Soil Plastic test</td><td>ASTMD422
ASTMD854-00
ASTMD2216
ASTMD4318
ASTMD4318</td><td>Initially and when soil quality changes</td><td>・Soil classifications: (Confirm specified soil material / particle size distribution curve / soil classification)</td></tr>
<tr><td>Soil unconfined compression test
Soil triaxial shear (compression) test
Soil consolidation test
Soil shear test
Soil permeability test</td><td>ASTMD2435
ASTMD2850 or D4767
ASTM D2435
Geotechnical society standards
ASTM D2434 or equivalent
reliable Standard / Specification</td><td>As necessary</td><td></td></tr>
<tr><td rowspan="4">Construction</td><td rowspan="2">Neces-sary</td><td rowspan="2">Site density or air void ratio / saturation measurements</td><td>JIS A 1213-4 or ASTMD1556</td><td>Embankment compacted either once every 1,000 m^3 or three times per 20 m of embankment length, whichever is higher.</td><td></td></tr>
<tr><td>Management guidelines for embankment compaction using an RI gauge</td><td>Standard construction area will be one layer per day. Control unit area will be set at 1,500 m^2. When the construction area is more than 2,000 m^2/day, the construction area is to be divided into two or more control units.
The following is a guide to the measurement points per control unit:<table><tr><td>Area (m2)</td><td>500 or less</td><td>500-1000</td><td>1000-2000</td></tr><tr><td>Measurement Pts.</td><td>5</td><td>10</td><td>15</td></tr></table></td><td></td></tr>
<tr><td rowspan="2">Other</td><td>Soil water content test</td><td>ASTMD2216</td><td>When water content change is noticed.</td><td></td></tr>
<tr><td>Cone index measurement</td><td>Handbook for pavement test methods</td><td>When trafficability is poor.</td><td></td></tr>
</table>

Section 3.4 Embankment for river bank

3.4.1 Outline of Embankment

The cross section shapes of river embankments shall be designed and planed after the consideration of characteristic of each soil condition based on past disaster history and other records.

It is important to carry out embankment under the suitable condition with such embankment material as adjusted in optimal construction water content and it must also be noted that the material quality for the embankment body will greatly impact upon the stability of the river embankment after completion and construction difficulty.

It is important to make the embankment body uniform and not leave harmful void for water tight. A partial defect will result in failure of the entire river embankment during a flood and destroy all the functions of the long embankment. So it is also important to construct the river embankment with both long section and cross sections of uniform strength.

3.4.2 Treatment of the foundation base

The main purposes of treatment of the foundation base prior to the embankment are as follows:

a) Improve the attachment of the embankment to the foundation base
b) Ensure smoother initial embankment work
c) Increase the bearing capacity by stabilizing the base
d) Prevent subsidence caused by decaying detrimental materials, such as plants

(1) Clearing and topsoil treatment

Constructing an embankment without clearing plants and stumps in the foundation base may loosen the embankment and cause harmful subsidence from the decay of these plants and stumps, affecting the stability of the river embankment. To prevent this, stumps, plants, and other obstacles such as stones or concrete blocks down to approximately one meter under the foundation ground level should be removed so that the embankment will firmly adhere to the foundation.

(2) Drainage treatment

If any puddles or water wells in the foundation are not dealt with before construction, the embankment cannot be compacted enough and the water may soak into the embankment, affecting the stability of the river embankment after construction. Water therefore needs to be drained carefully.

(3) Treatment for unevenness

Uniform and even quality is required for embankments. But where having a gap or uneven foundation bases, the compacting depressed area and uneven areas would be insufficient and not create a uniform embankment, and it will also affects embankment smoothing work. For this reason, it is necessary to smooth out any unevenness as much as possible prior to embankment work so that the finished embankment is uniform.

3.4.3 Embankment and Compaction

Water resistance is the focus for river embankment construction work. Therefore, it is important to select construction methods that make the embankment body uniform.

(1) Spreading

The fill material brought by transport machines is to be spread evenly for compaction. Spreading is the most important work for uniform compaction. Carefully spreading a thin layer of fill material enables construction of an even and well compacted embankment.
Though this spreading work may not seem important for embankment construction, it actually has the large impact on the quality of the embankment.

An embankment spread at a defined and even thickness is uniform and stable. On the other hand, an embankment with a spread that is too thick is insufficiently compacted, possibly leading to consolidation subsidence of the embankment itself or differential settlement in the future. Therefore, it is important to understand that the utmost care must be taken in spreading work for embankment construction.

<u>Notes regarding machinery</u>

As bulldozers are continuously excavating, hauling, and spreading, it is sometimes difficult to confirm the layer thickness and care needs to be taken to avoid the layer becoming too thick. In general, the thickness of the river embankment after compaction is defined less than 30cm and in the TS and SSoRM, it is stipulated less than 23cm per layer. In such cases the spread thickness (thickness before compaction) is the thicker one than such thickness reduced by compaction. Care needs to be taken so that it does not become too thick.

<u>Notes regarding manual labor</u>

When manually constructing an embankment, the thickness of the spread must be less than 15 cm. No soil mass larger than 10 cm should be in the spreading materials, and soil mass that is larger than this should be broken into smaller pieces in advance.

Notes regarding cohesive soil containing a high water content

When using cohesive soil containing high water content as fill material, ruts can easily be created by haul vehicles and the soil strength is significantly lowered from remolding. In order to prevent this, create other transportation routes or have secondary transportation to embankment locations close to transportation routes.

Secondary transportation will also spread the fill material. It involves using a spreading machine to press soil from the unloading point on the transportation route to the embankment site. As the length for this spreading work is longer than usual, bulldozers with small ground contact pressure are generally used.

(2) Compaction Work and Machinery

Whether compaction work should be performed by construction machinery or manual labor is decided through comprehensive consideration of the construction scale, importance of structures, on-site conditions, flexibility of compaction machines, and economic efficiency compared to the price of construction.

Understanding the effects of trial construction (spread thickness, number of compaction passes, and water content) as much as possible, it is important to secure the embankment body by compacting to a specified level of quality. The following points need to be considered for construction:

a) Compact the entire embankment evenly. Carefully compact the ends and corners of the embankment to ensure sufficient compaction.
b) Carefully drain during embankment construction, accounting for transverse gradients. When rain is forecast, smooth the soil surface so that rain does not pool or soak into the soil.
c) Water content impacts mostly the compaction works and the result of compaction degree of the compacted embankment. It is important to check the water content and to take action for suitable water content of the fill material during the construction.

Notes regarding manual compaction

(a) As manual compaction is low energy, the spread thickness should be less than 15 cm per layer when completed.
(b) To meet the construction deadline, the required number of workers should be

calculated based on the target work amount per day. Whether the necessary number of workers can be procured must be confirmed during planning.

(c) To ensure a specified level of quality, small compaction apparatuses that can be used manually, such as seven-kilogram rammers, plate compactors or hand-roller, are preferable.

(d) When using a tamping apparatus, a trial construction should be conducted in advance to determine how many times each soil layer needs to be tamped in order to achieve the desired level of quality (degree of compaction on-site).

(e) As spread thickness will differ depending on the soil or combination of support equipment (rammers, roller etc.), an appropriate thickness should be set through trial construction in advance.

<u>Notes regarding compaction using heavy machinery</u>

The heavy machinery to be used for compaction is to be selected based on construction conditions, such as soil quality of fill material, type and scale of construction, and the characteristics of compactors. Soil quality is a particularly important selection criteria. Fill materials vary widely from crushed rocks, sandy soil, and cohesive soil containing a high water content. Moreover, the adaptability to compaction often differs greatly even within the same soil according to water content. At the same time, the compacting functions will vary depending on the model of compactor, and the compaction effect varies even with the same model depending on standards and performance, including size, weight, linear pressure, tire pressure, frequency, vibratory force, impulse force, and travelling performance. It is important to understand these points well when selecting machinery to compact the soil effectively.

For river embankment construction work, soil quality used to vary based on how the soil is obtained, with moist soil being used frequently. Also, conventional bulldozers were often used as they allow spreading and compaction to be performed with the same machine. In case of use of purchased fill material, different machinery is to be used according to quality of the soil to be compacted in order to increase river embankment stability. Reference table 3.4.1 roughly indicates the machinery selected for general compaction work and outlines the work features of major compactors.

Reference table 3.4.1 General adaptability of soil and compaction machinery

Machine category / Main soil	Regular bulldozer	Tire roller	Vibrating roller	Vibrating compactor	Tamper	Notes
Sand Sand with gravel	○	○	○	○	○	Sand of one particle size, broken Gravel, desert sand
Sand, sandy soil Cohesive soil with gravel	◎	◎	○	○	○	Fine soil and pit gravel with an optimal particle ratio for compaction
Cohesive soil Cohesive soil with gravel	○	○	○	×	○	Soil containing many small particles with less sensitivity, loam soil with low water content, hardpan that can be easily broken
Sandy soil with high water content Cohesive soil with a high water content	○	×	×	×	×	Soil difficult to adjust water content to obtain trafficability, silt soil

◎ : Effective

○ : Can be used

○ : Can be used where other apparatus cannot be used due to construction scale

× : Inappropriate

(Quoted from "River Embankment manual")

(a) Compaction using bulldozers

Bulldozers are often used as compactors for river embankment construction. In such cases, careful construction is required so that the embankment does not become rough. Depending on the fill material, a decrease in spread thickness must also be considered for more effective compaction.

The river embankment should ideally be compacted along its parallel line. It is necessary to always ensure that the compaction width can be overlapped during compaction.

Photo3.4.1 Spreading & compaction by bulldozer

(b) Compaction using tire rollers

This type of compaction takes advantage of tires filled with air. Tire contact pressure on the ground can be changed through loaded weight and air pressure. Tire pressure is directly related to compaction; generally a higher ground contact pressure is used to compact crushed stones, and a lower pressure is used to compact cohesive soil.

When using tire rollers, care must be taken to track conditions such as tire load and pressure during compaction as these could result in great difference in the effect of compaction.

Photo3.4.2 Compaction by tire-roller

(c) Compaction using vibrating rollers

A vibrating roller combines a vibration generator with a roller to obtain high compaction with a small weight by aligning soil particles through vibration. Vibrating rollers are known to be effective for compaction of gravel and sandy soil, which are generally less cohesive. Appropriate selection of the weight of the roller and vibration frequency is necessary.

Small vibrating rollers have seen wide use in the past. Large vibrating rollers are more effective than other machines at deep compaction, enabling increased spread thickness.

Conversely, vibrating rollers tend to slip and are unable to function on rocks, soil mixed with rock pieces, and sand with the same particle size. This must be taken into consideration.

(d) Compaction using vibrating compactors and tampers

Vibrating compactors and tampers compact by directly attaching the machine to flat plates using vibrations. Due to their light weight, they can be used in places where other machines cannot, such as around buildings, the top and slopes of embankments, as well as in small scale and manual compaction.

Photo 3.4.3: Compaction using a hand-roller, Plate vibrating compactor & tamper

3.4.4 Checking quality of embankment

Measure the characteristics (compaction degree, air void ratio and others) that show quality, make sure that these results sufficiently meet the quality standard by attending the inspection and checking the test record, and check if work was carried out consistently by using control charts.

When quality does not meet the standard values or work was not consistently carried out according to the daily management results, review to see if work standards are being followed and there are no errors, and make appropriate corrections. If quality varies drastically from standard values, on-site corrective action, such as re-compaction, is necessary. Particularly if weather is the cause, stop work conditions must be reassessed.

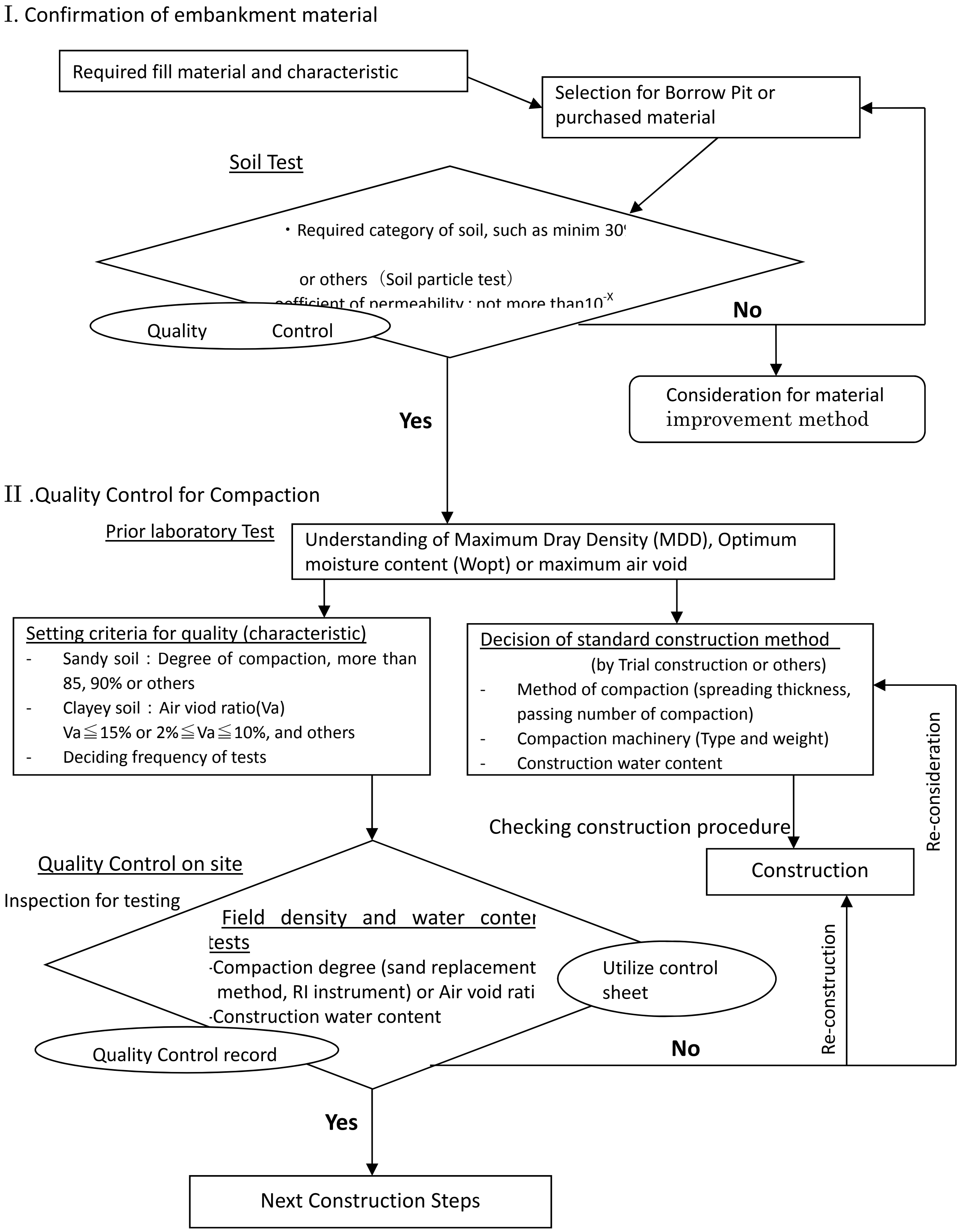


Figure 3.4.2 Procedure of Quality Control for embankment

3.4.5 Quality Control for Compaction

Compaction management should follow the criteria and compaction frequency described in the Specifications and the Contract Documents. The following criteria and frequency are used as reference values when specifications are not defined.

1) Quality control test by Sand Replacement Method

Sand Replacement Method is conducted by the provisions defined in the specifications of ASTM D1556 (AASHTO T191), BS 137 or JIS 1214. The criteria and frequency of the tests are as follows;

(a) Applicable grain size

Applicable soil for Sand Replacement method shall have the maximum its article size with 53mm. The soil with the maximum particle more than 53mm will be conducted in other test method such as “Strike Sand method”.

(b) Criteria of the Tests

Criteria of the test are to be in accordance with followings by its characteristic factors unless otherwise specified in the provisions or specifications in the Contract Document.

i) Criteria of “Degree of Compaction”

Degree of compaction can be obtained by the ratio between the maximum dry density (MDD) previously determined by indoor compaction test (ASTM D698/D1557 or JIS A1210) and the field density (FD) by using the sand replacement method (ASTM D1556 or JIS A1214) on materials compacted at the site on fill materials as below formula;

$$\text{Degree of compaction} = \frac{\text{Dry density (FD) of fill materials compacted at the site}}{\text{MDD from indoor compaction test}} \times 100\ (\%) \quad \cdot\cdot\ (1)$$

The criteria of Degree of Compaction will be decided according to nature of proposed embankment material and strength required by design for each Works, within the range between 85% ～90%.

II) Criteria of “Void ratio of air” or “Saturation rate”

In case it is difficult to test based on above mentioned “Degree of Compaction”, a method using “Void ratio of air” or “Saturation rate” may be allowed for test.

When sandy soil (25%≦passing through 75 μ sieve<50%)

Void ratio of air Va: Va ≦ 15%

When cohesive soil (50%≦passing through 75 μ sieve)

Saturation rate Sr: 85% ≦ Sr ≦ 95% or

Void ratio of air Va: Va ≦ 15%

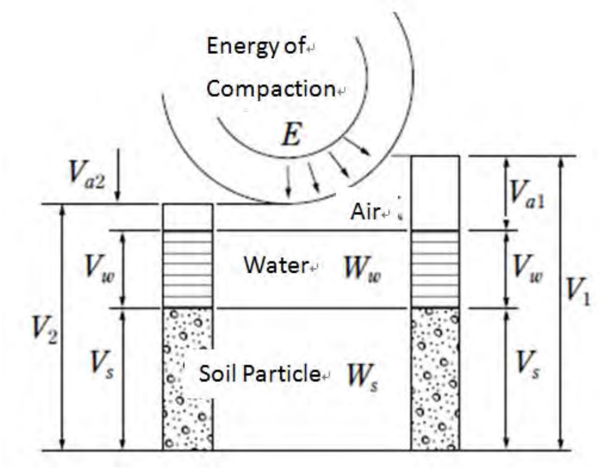
(c) Frequency of Sand Replacement Test

On an embankment, test is conducted by the frequency of either once every 1,000 m^3 or three times per 20 m of embankment length, whichever is much times.

Basically the test result of one (1) place (or one test) is evaluated by the average of three data measured from three (3) holes of Sand Replacement method for one (1) place. Other matter regarding frequency of test shall be carried out by the instructions of the Engineer.

2) Definitions of Characters for Compaction

Soil is consisted by soil article, water and air and its image of proportion of content are illustrated as below figure 3.4.2;



Specific Dry Density

$$\gamma_d = \frac{W_s}{V_2} \quad \text{.......} \quad (2)$$

Water content

$$w = \frac{W_w}{W_s} \times 100\ \% \quad \text{.......} \quad (3)$$

Figure 3.4.2 Proportion of soil structure (image illustration)

By the compaction energy, such as rolling by compactor, soil will be consolidated and water and air will be exhausted. Then soil will increase its density with crushing its particle and changing its alignment. The enough compacted soil is not likely to be soft against seeping water and become larger in its strength and bearing capacity, and is also to decrease compressibility.

Definitions of characteristic factor used in the quality control of compaction, such as dry density and degree of compaction, are as follows;

$$\text{Void ratio of air:} \quad Va = \frac{Va}{V} \times 100\ (\%) \quad \ldots.. \quad (4)$$

$$\text{Saturation rate:} \quad Sr = \frac{Vw}{Vv} \times 100\ (\%) \quad \ldots.. \quad (5)$$

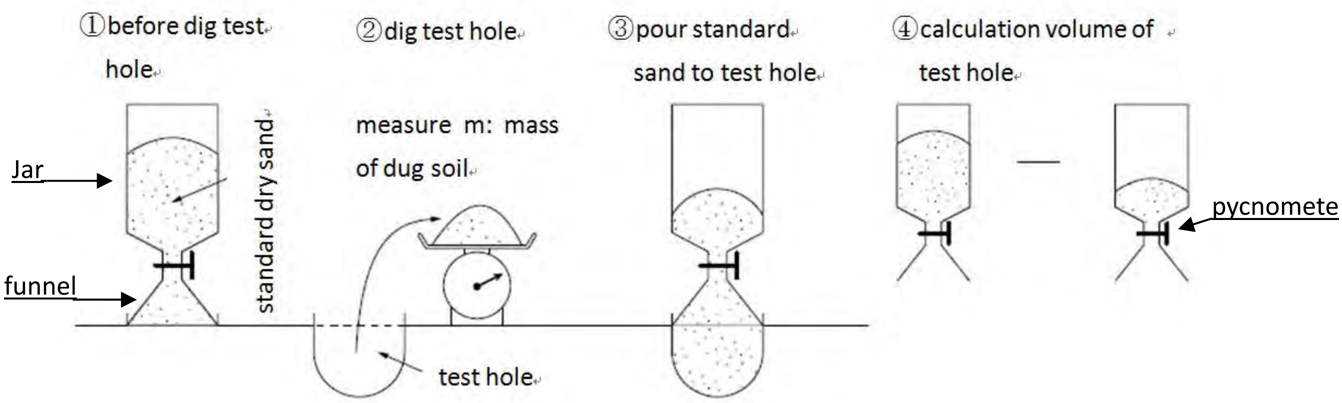
Here, Vv = Va + Vv : Volume of the part of void

3) Introduction and Notes of various testing methods

The detail of Sand Replacement Method and introduction of other testing methods are as below;

(a) The detail of Sand Replacement Method

Sand Replacement Method is tested according to the standards described in ASTM or JICA as sub-clause 1) above. And Sand Replacement is carried out by the measuring the volume of proposed soil space replaced by standard sand with previously known its specific gravity as shown the below figure and photos;



④ Volume of test hole = mass of filled sand in hole / known specific gravity of standard sand

Figure 3.4.3 Measuring idea for Sand Replacement

Example Data sheet and calculation for field density (Compaction Degree) are shown on Table3.4.1;

Table3.4.1 Data sheet and calculation for compaction degree

Density Test for soil by sand replacement method (JIS A 1214)

Project Name: Date of test:
Location of tests: Name of tester:

Testing apalatus	No.			Name of Soil		
Dry density of using standard sand	ρ_{ds}	g/cm^3		Mass of sand fully filled for the funnel		
No. of test hole			1	2	3	Remark
Largest particle size		mm	*37.5*	*37.5*	*37.5*	
No. of container			1	2	3	
Mass of the container		g	*232.3*	*236.4*	*232.1*	
Mass of (the soil extracted from the hole + the container)		g	*3171.8*	*3097.6*	*3084.5*	
Mass of the wet soil extracted from the test hole	m_7	g	*2939.5*	*2861.2*	*2852.4*	
Mass of the soil, after dried by furnace, extracted from the test hole	m_0 = 100 m_7 /(w +100)	g	*2679.6*	*2606.5*	*2593.5*	
Mass of sand filled with jar and pycnometer	m_3	g	*8550.2*	*8491*	*8461.5*	
Mass of remaining sand and measuring apalatus	m_8	g	*5258.4*	*5234.6*	*5067.4*	
Mass of sand which enters into test hole and funnel	$m_9 = m_3 - m_8$	g	*3291.8*	*3256.4*	*3394.1*	
Mass of sand need to fully fill with test hole	$m_{10} = m_9 - m_6$	g	*1936.8*	*1901.4*	*2039.1*	
Volume of test hole	$V_0 = m_{10} / \rho_{ds}$	cm^3	*1501.4*	*1474*	*1580.7*	
Wet Density	$\rho_t = m_7 / V_0$	g/cm^3	*1.958*	*1.941*	*1.805*	
Dry Density	$\rho_d = m_0 / V_0$	g/cm^3	*1.785*	*1.768*	*1.641*	
Water Content: No. of container			1	2	3	
Water Content: Mass of sample and container	m_a	g	*1273.8*	*1299.5*	*1333.8*	
Water Content: Mass of draied sample and container	m_b	g	*1184.8*	*1209.0*	*1239.8*	
Water Content: Mass of container	m_c	g	*269.3*	*283.0*	*298.3*	
Water Content: Water Content	W	%	*9.7*	*9.8*	*10.0*	
Average: Water Content	W	%	*9.8*			
Average: Wet Density	ρ_t	g/cm^3	*1.901*			
Average: Dray Density	ρ_d	g/cm^3	*1.731*			
Compaction Degree	$C_d = \rho_t / \rho_{max}$	%	*92.8*			

Notes;

1. Maximaum Dry Density: ρ_{max}= *1.866* g/cm^3 (Previously obtained at Laboratry test)

2. Water Cintent: W = $(m_a - m_b) / (m_b - m_c)$ = % *100×(1273.8 - 1184.8) / (1184.8 - 269.3) =* *9.72*

3-1. Mass of sand in funnel m_6 = *$m_9 - m_{10}$= 1355 g* 3-2. Density of standard sand : *ρ_{ds}=1.2899 g/m^3*

4. *Inclined figures in violet color: Example calcultation for Compaction Degree using the sheet*

Photo 3.4.4 Field density test (sand replacement method)

(b) QC by RI (Radio Isotope) apparatus

A Radio Isotope (RI) apparatus ,or gauge, is suitable for daily management because measurements are easy to take and it shows the results at the site. This method can be applied to fill material that is less than 100 mm in maximum particle diameter. It measures the dry density of fill material that was compacted by using an RI gauge and obtains the degree of compaction by formula (1) listed above. The maximum dry density (MDD) needs to be obtained in advance by using an indoor soil test.

Photo 3.4.5 Field density test using an RI gauge

(c) Control by Proof Rolling

Proof rolling is used to check embankment consolidation by having heavy machinery slowly drive on the completed embankment surface and visually check ground settlement and the tire rut. The test itself is quite simple, but the inspector needs experience and judgment as it is determined visually.

(d) Alternative Method for testing field water content (Frying pan-method)

As it is shown on the formula (3) (w=Ww/Ws*100%) in the above sub-clause 2) Definitions of Characters for Compaction, the water content of soil shall be described % of proportion between "weight of water" and "weight of soil particle" in the lump of soil. This construction water content greatly affects the workability of compaction and the quality of embankment after compaction work. Methods of water content testing are stipulated in the standards of ASTM D2216 and JIS A 1203 and other Standards.

But those ordinary methods by such standards have to wait at least 24 hours after drying test sample at 110℃. However the accuracy is a little down, there are also some water content testing methods with easiness and taking shorter time to test. "Frying pan-method" using ordinary kitchen cooking stove and frying pan are here introduced as well as the micro wave- method being used on site.

○ Equipment and method of "frying pan-method"

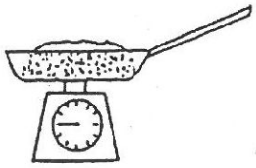
Equipment and method of "frying pan-method" are as follows; "Frying pan-method" for field soil water (or moisture) content test was used in accordance with ASTM D4959 (ASTM 2000).

The frying pan serves as the specimen container and the mass of the empty pan and the wet mass of the soil sample are measured before drying. The frying pan is placed on the burner like cooking on a conventional stovetop and sample is stirred while heating.

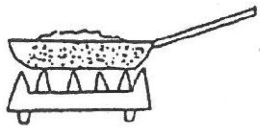
The specimen and pan are removed from the heat and weighted at 1-min intervals. The process continues until a change in soil mass of less than one percent occurs during the 1-min interval, at which point the moisture content is calculated (measure dry mass of soil).

the water content is calculated as below and shown on the photos;

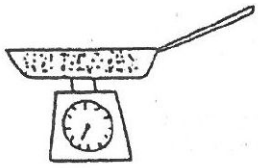
(a) Weigh the mass of wet sample + frying pan (100 – 500 gramme)



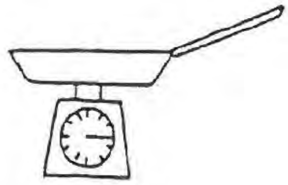
(b) Dry the material on burner or stove



(c) Weigh the mass of dry sample + frying pan



(d) Weigh the mass of frying pan



$$\text{Water (Moisture) Content} = \frac{(a) - (c)}{(c) - (d)} \times 100 = \frac{\text{mass of water}}{\text{mass of dry soil}} \times 100 \quad (\%)$$

Photo 3.4.6 Measuring field water content (Frying Pan-Method)

e) Common items for density test methods

When either the sand replacement method or an RI gauge is used, make sure to re-compact when the degree of compaction does not meet control standards.

When using fill materials such as cohesive soil that lower bearing capacity from remolding, make sure there is no excessive compaction. In order to prevent this from occurring, compaction on the test embankment needs to be carefully observed to decide the compaction machinery and construction methods.

4) Data sorting

Measured data is to be sorted every day and shown on a progress capability chart, which shows changes in quality in conjunction with construction progress and is convenient in predicting abnormal values and investigating causes together with the daily management status.

(1) Process capability chart

As seen in Figure 3.4.3, a process capability chart shows values of quality characteristics (such as "degree of compaction") on the vertical axis and sampling numbers on the horizontal axis. It continuously catches data fluctuations of standard values by recording measured values temporally and displaying upper and lower limit standard values for these values. Therefore, when looking at a process capability chart it is necessary to carefully watch for values outside of the standard, how the dots are aligned and the range of variation.

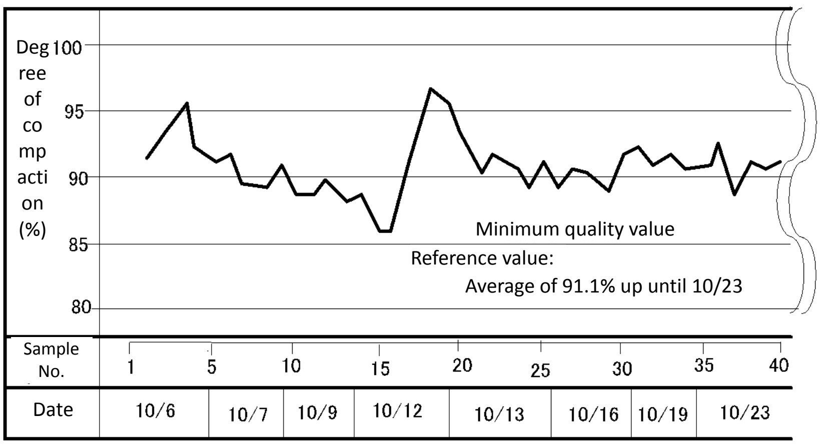


Figure 3.4.3 Example of process capability chart

(2) Data usage and processing

It is difficult to apply a strict management chart for embankment quality control because of changes in the soil and water content of the fill material, different compaction methods depending on the embankment location, and a small number of documents for quality control tests. Therefore, when abnormal variations are observed by plotting measured data, it is important to investigate the cause and take the necessary action. In particular, suitable action must be taken for changes in fill material and water content, reviewing the appropriateness of construction methods and machinery if necessary.

3.4.6 Trial Construction for compaction

Prior to the commencemet of main construction for river bank embankment, Trial Construction for embankment shall be cariied out in order to define the standard procedure of embankment works and the criteria for construction control. Basically Trial Construction will be conducted under the same fill material and the same construction condition as actual main construction.

(1) Trial Construction for deciding standard procedure of embankment works

As shown on Figure 3.4.2 “Procedure of Quality Control for embankment”, Trial Construction shall be carried out and the following construction factors and quality control values are measured in order to determine the standard method and procedures satisfied the quality requirements at the time of commencement of the control of compaction quality;

- Method of compaction (thickness of spreading, numbers of compaction passes and others),
- Compaction machinery (type and weight),
- Construction water content and others

The most suitable combination of such construction factors is to be decided and it becomes a standard method of embankment works of the Project.

At the Trial Construction for embankment, one of example of combination of construction factors is shown the table below;

Table 3.4.2 Example of combination of thickness of spreading and numbers of compaction passes

Passes / Thickness Per one (1) layer	compaction passes per one (1) layer by compactor		
	2 times	4 times	6 times
20cm	Degree of compaction	Degree of compaction	Degree of compaction
25cm	Degree of compaction	Degree of compaction	Degree of compaction
30cm	Degree of compaction	Degree of compaction	Degree of compaction

(2) Trial Construction for Control by standard construction procedure

The standard construction procedure was decided by satisfaction with required quality requirement, such as required degree of compaction and others, in Trial Construction. In Control by standard construction procedure, actual embankment works shall be conducted according to the defined standard construction procedure. Items and construction factors at Trial Construction are the same as stated in sub-clause (1) above.

(3) Example of one of Trial Compaction for river embankment Works

Report for Trial Compaction for Embankment works

18 March, 2016

SV Team / JET and BWDB Moulvibazar O & M Division

Name of Project: Capacity Development of Management for Sustainable Water Related Infrastructure, BWDB / JICA

Name of Works: The Pilot Repair Works of Manu River Flood Control Embankment in Moulvibazar District

In order to consider the construction method for compaction, Trial Compaction for embankment works, Step1, has been carried out on 08, 09 and 10 March 2016. And the same of Step3, using Bulldozer, was carried out on 13 March 2016. The trial result and recommended method of the works are reported herewith. According to the recommendation, embankment works are to be implemented on the site.

1. Objective and procedure of Trial Compaction for embankment works:

1-1: **Objective of Trial Compaction for embankment works**

The objective of Trial Compaction is to determine the proper and adequate construction method in order to keep the required quality by the Specifications and Drawings. Trial Compaction will be carried out and the following construction factors and quality control values are measured in order to determine the standard method and procedures satisfied the quality requirements at the time of actual embankment construction;

Method of compaction:

- **thickness** of spreading of material, numbers of compaction **passes** and
- Compaction **machinery** or equipment (type and weight),
- Construction **water content** and others

The most suitable combination of such construction factors is to be decided and it becomes a standard method of embankment works of the Works.

1-2: Step1, 2 & 3 procedure of Trial Compaction

Trail would be divided into three steps;

Step1. Using a relatively small compaction equipment (Plate Compactor) suit to the site

Step2. Checking manual compaction (by only 7kg hammer) in same condition of Step1.

Step3. In case Step1 is not attained satisfactory result or when compaction machinery is to be used on the site, Trial Compaction will be carried out using such machines.

Procedure of Trial Compaction for Step1.

a) Characteristics of proposed embankment material

Proposed material's Maximum Dray Density (MDD) and Optimum Water Content are shown on below table;

Table 1. Summary of test result of Laboratory Proctor Test

Name of Test	The location of Sampling								
	Borrow Pit No.1			Borrow Pit No.2			Existing bank		
	A	B	Aver-age	A	B	Aver-age	A	B	Aver-age
Optimum Moisture Content Wopt (%)	19.25	18.54	18.90	18.65	18.68	18.67	21.00	21.48	21.24
Maximum Dry Density MDD (kN/m3)	16.18	16.38	16.28	16.58	16.18	16.38	16.21	16.33	16.27

*kN/m3 = 0.102 g/ cm3

b) Trial procedure for Step 1. (Using 110 kg weight "Plate Compactor")

Trial Compaction will be carried out in two different thickness and three different numbers of compaction passes, 6 different procedure of compaction are carried out as below;

Table 2. Combination of thickness of spreading and numbers of compaction passes

Passes by compactor / Thickness Per one (1) layer	compaction passes per one (1) layer by compactor		
	2 times	4 times	6 times
20 cm	a) DC	b) DC	c) DC
30 cm	d) DC	e) DC	f) DC

Testing parameter DC: Degree of compaction (% of maximum dray density (MDD)

c) Procedure of each test

i) Spread embankment material until required thickness (loose) for one layer

ii) Compacting each layer of 3m length and 3 m wide area in different numbers of passes of compaction by Plate Compactor.

iii) Check settlement, height of compacted thickness, after two passes compaction.

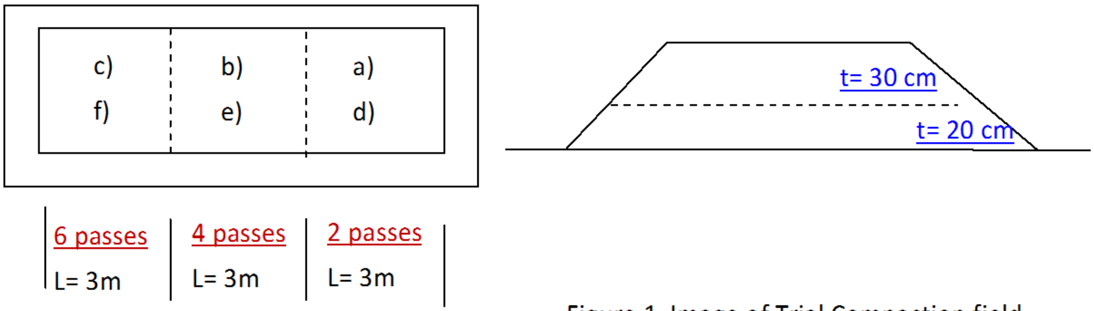


Figure 1 Image of Trial Compaction field

iv)

After compaction, field density test will be conducted with each two (2) holes by sand-replacement method. Each test (holes) will be also checked its water contentment.

2. <u>Record/result of Trail Compaction, Step 1.:</u>

Basic conditions:

1) Material from borrow pit No.1
2) Optimum Moisture Content
   Wopt : 18.9 (%)
3) Maximum Dry Density
   MDD: 16.28 (kN/m3) = 1.66 (g/cm3)
4) Compaction by Plate Compactor
   (110kg model, refer to photo right)

2-1: Spreading thickness 20cm, in different nos. of passes by the compactor

Tests results of first layer of 20cm thickness are shown on below table;

Table 3. Summary of test result of Field Density for 20cm layer

Name of Test and out-comes	Field Density (g/cm3)			Water Content (%)			Degree of Compaction ; DC (%)			
No of holes / Numbers of passes	1	2	3 .	1	2	3 .	1	2	3.	Average
2 passes	1.357	1.438	1.397	25.0	17.6	17.5	81.7	86.7	84.1	84.2 84.2
4 passes	1.359	1.450	1.409	20.0	17.6	19.9	81.8	87.4	84.8	84,6 84.7
6 passes	1.449	1.550		19.0	14.3		87.3	93.4		90.3
Remarks			on 09 Mach'16						on 09 Mach'16	

Note: 1) kN/m3 = 0.102 g/ cm3 2) No.3 hole of 2 & 4 passes are additionally tested one day after No.1 & No.2
Average of DC: upper is Av. Of No.1 & No.2, lower is Av. of 3 holes

2-2: Spreading thickness 30cm, in different nos. of passes by the compactor

Tests results of first layer of 30cm thickness are shown on below table;

Table 4. Summary of test result of Field Density for 30cm layer

Name of Test and out-comes	Field Density (g/cm3)			Water Content (%)			Degree of Compaction ; DC (%)			
No of holes / Numbers of passes	1	2	3 .	1	2	3 .	1	2	3 .	Average
2 passes	1.508	1.456		24.3	23.1		90.9	87.7		89.3
4 passes	1.520	1.390		21.3	24.1		91.8	83.8		87.8
6 passes	1.590	1.330		24.1	23.5		96.1	80.3		88.2

Note: 1) No.2 holes of 4 & 6 passes are tested from the surface 5 cm below the compacted surface level, after 5 cm compacted soil was taken away.

After No.1 hole test, with satisfactory result of DC 91.8 %, it is tried to test in other conditions for No.2 hole in order to find the difference of test results (compaction degree) between the test's hole from the compacted surface and the test's hole from 5 cm below the compacted surface.

3. <u>Findings from the results and recommendation of the method of the works</u>

3-1. Relation between water content and field density (FD)

There is no particular finding of differences of FD according to water content.

Relation between water content and field dry density of the test is shown on Figure 1.

However this time test results, construction water content is desired to be as possible as near as optimum water content (Wopt) according to the general theory of compaction of the soil.

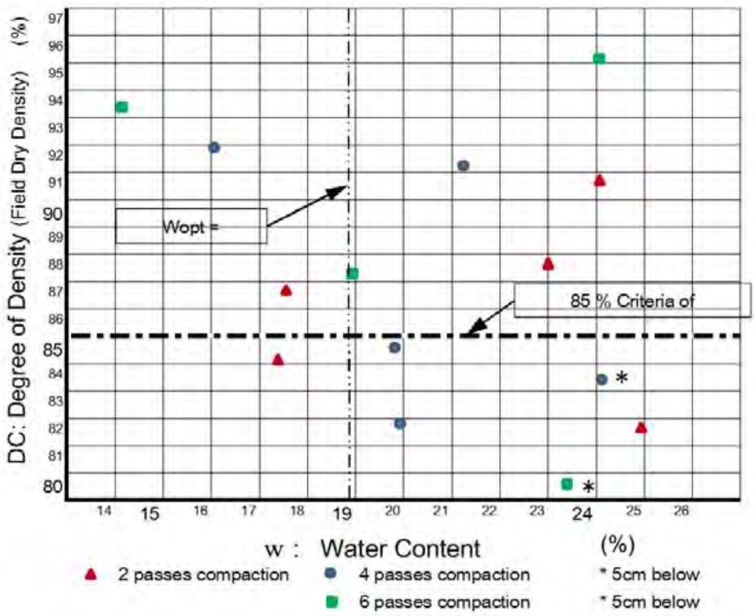


Figure2. Relation between water content and Field Dry Density

3-2. Relation between spreading thickness and field density

There is no particular finding of differences of 20 cm thickness of loose spreading layer 30 cm thickness of the same.

But as additional study of checking of field density of in-side body of the embankment, with 5cm taking out from the top of compacted embank in 4 passes and 6 passes of 30 cm thickness layer, it is found apparently compaction is not effected with more depth from the surface of compaction.

3-3. Relation between number of passes of compaction and field density

There found that the more passes makes embankment the more strong, more compacted.

Compare with the averages of 20 cm thickness layer and each No.1 holes results of 30 cm thickness layer apparently shows 6 passes compaction is more satisfactory than others and 4 passes compaction is more effective.

3-4. Recommendation of the method of the works

According to above findings by Trial Compaction and taking consideration of the site condition, following recommendation is made for the Quality Control for actual embankment works;

1) When using Plate Compactor, numbers of passes of compaction is preferably 4 times.
   But after actual Degree of Compaction (DC) is good enough, the times of compaction may be decreased provided by obtaining DC more than requirement.
2) Thickness of spreading material is preferable not more than 20cm. In case of obtaining satisfactory DC result, it may increase, but it is to be not more than 30cm.

4. <u>Record/result and Findings/recommendation of Trail Compaction, Step 3:</u>

4-1. Basic conditions:

1) Material from borrow pit No.1 & No.2
2) Optimum Moisture Content
   Wopt : 18.78 (%)
3) Maximum Dry Density
   MDD: 16.23 (kN/m3) = 1.655 (g/cm3)
4) Compaction by Bulldozer
   (5 ton, D3 CAT Caterpillar)

4-2. Initial plan for testing Step3

After preparation of base for test yard, 25cm thickness of material is spread in loose by bulldozer or manual labour.

Number passes of compaction by bulldozer and associated one/more passes compaction by Plate Compactor was initially planned for Step 3 Trial as below sketch;

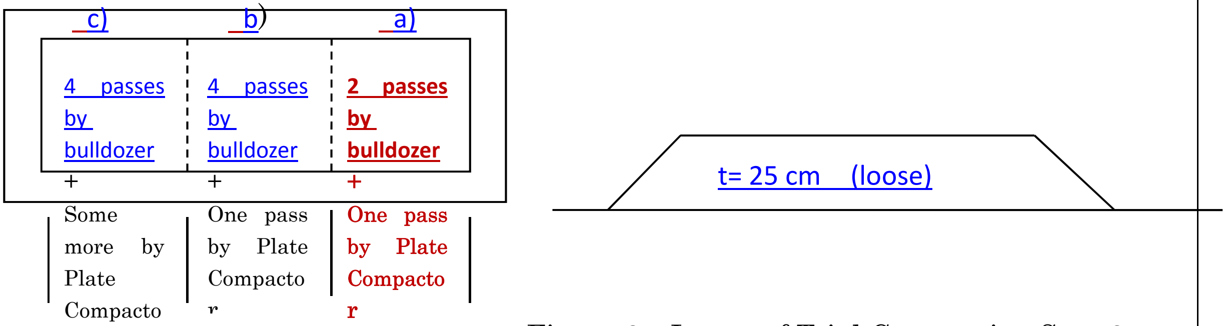


Figure 3 Image of Trial Compaction Step 3

4-3. Record/result of Trail Compaction by bulldozer, Step 3

1) After good result of test a), test b) was changed in the condition of 2 passes compaction by bulldozer only (no extra compaction by Plate Compactor).

Test results are shown as below table;

Table 5. Test result of Field Density for 25cm layer compacted by bulldozer

Name of Test and out-comes			Field Density (g/cm3)		Water Content (%)		Degree of Compaction ; DC (%)		
No of holes / Numbers of passes			1	2	1	2	1	2	Averag
By bulldozer		By Plate Compactor							
a)	2 passes	1 Pass	1.61	1.62	15.8	18.5	97.3	98.1	97.7
b)	2 passes	nil	1.58	1.59	15.6	19.3	95.5	95.8	95.6
Remarks							Good enough to over 85 %		

Note: 1) Test b) were conducted are tested from the surface 5 cm below the compacted surface

2) Test c) were cancelled because of no need

2) Reference information of the thickness contracted by compaction

By supplementary checking the thickness of layer after compaction by 2passes of bulldozer compaction, it is roughly found about 3- 4cm shrinkage from the 25 cm loose spreading material layer.

4-4. Findings/recommendation of Trail Compaction by bulldozer, Step 3

According to above results of Trial Compaction test and taking consideration of the site condition, following recommendation is made for the Quality Control for actual embankment works;

1) When using Bulldozer's spreading & compaction of material layer by layer, numbers of passes of compaction is 2 times enough.

But in case of actual Degree of Compaction (DC) is not good enough, the times of compaction should be increased or other supplementary means must be taken.

2) Thickness of spreading material is preferable not more than 25cm. In case of obtaining satisfactory DC result, it may increase, but it is to be not more than 30cm.

Attachments

Photos of Trial Compaction for embankment works

<u>Photos of Trial Compaction</u>

Photos: Trial Compaction for embankment Step 1 & 3 taken on March, 2016

	
Ph A. Preparation for testing yards (backfilled existing Pond)	PhB. Step1(using "Plate Compactor"); compaction and check thickness after 2 passes of compaction

PhC. Field Density (FD) Test by sand-replacement method for 2 passes compaction

PhD. Check the spreading thickness of 30cm.

PhE. FD test of no.2 hole for 6 passes but from 5cm below the compacted surface.

PhF. Preparation for testing yards (from beginning point of down-stream, about 30m at 6.2m PWD)

PhG. Spreading material by bulldozer by layer by layer, until loose thickness 25 cm

PhH. Field Density (FD) Test by sand-replacement method for test a) area ; 2 passes compaction by bulldozer plus one pass compaction by Plate Compactor

Section 3.5 Earth works on Slope

In general, compaction is insufficient on the crest and slopes of embankments; embankment slopes must be compacted thoroughly as well to satisfy the designed cross section. When the compaction on slope surfaces is insufficient in comparison with the entire embankment, they are often destroyed by heavy rain. In order to avoid these collapses, slopes are desired to be compacted with machinery to ensure sufficient compaction.

Slope erosion due to rainfall is an item to be careful of during construction. In particular, runoff concentrated on one area of the slope is a main cause of slope erosion and may result in defects on the completed slope. Even during construction, it is therefore important to build provisional ditches at appropriate spacing to let the rainfall drain. The construction method generally used, shown in Figure 3.5.1, prevents runoff from concentrating on one area by having a gradient of a few percentage transverse to the embankment.

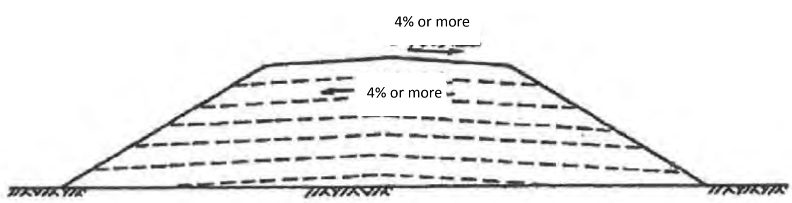


Figure 3.5.1 Protecting the slope during embankment construction

Section 3.6 Earth works accompanying structure construction

Sections of sluice gates that connect river structures and embankments often tend to create different levels through differential settlement, resulting in cracks and leaks.

Different levels are often seen at the foundation base of embankments that are soft. Causes of subsidence in the areas that connect embankments and structures are subsidence of the foundation base, consolidation subsidence of the embankment itself, and changes in structure position from embankments behind the structure. However, other construction methods may also cause subsidence. Specifically, as structure and embankment construction proceed in parallel when a new river embankment is being built, in many cases the embankment is attached to the structure when the structure and the river embankment are almost complete. The following problems are thus possible causes of subsidence at the contact section:

a) The embankment at the connecting area tends to be thicker as is the last to be constructed, and the construction area is narrow so compaction is often insufficient.
b) The embankment at the connecting area is often surrounded by structures, such as tall sluice gates and embankments, so drainage may be insufficient.

For these reasons, the following points should generally be considered when constructing embankment at the connecting area between embankments and structures:

(a) Select stable materials for the embankment at the connecting area with a certain degree of water shielding that are easily compacted and do not weaken when exposed to water.
(b) Select appropriate compactors for thorough construction work to avoid insufficient compaction in limited and narrow work areas.
(c) As water tends to concentrate around the backfill of structures during and after construction causing settlement and collapses, soil selection and drainage measures, such as ensuring sufficient drainage gradient during construction, should be taken.
(d) The concrete surfaces of the intersection of concrete structures and materials for embankment at the connecting area need to be damp prior to filling to avoid changing the water content ratio of the fill material.
(e) As heavy machinery cannot sufficiently compact around walls, small machinery should be used for sufficient compaction and the spread thickness should be thin for more effective compaction.
(f) Ensure that the embankment at the connecting area does not apply earth pressure

before the structure exhibits sufficient strength.

(g) Ensure that uneven earth pressure is not applied even after the structure exhibits strength by compacting evenly from both sides of the structure.

Attention should also be paid to the following cases:

<u>Soft ground</u>

Preloading prior to constructing structures should be considered if the foundation base consists of soft ground.

<u>Expansion work for river embankment</u>

In many cases, structures that accompany expansion work for river embankment are built by excavating an existing river embankment, meaning that preloading from the existing river embankment will keep subsidence below that of a newly built river embankment. However, as new embankments will be constructed in an expanded area, they will subside the same as a new embankment, frequently resulting in differential settlement. Therefore, preloading must be considered for expansion areas in the same way as new construction if the ground is soft.

It is also necessary to thoroughly investigate structures and the surrounding embankment, and make repairs or fill any cavities if there are deformations when making extensions to structures in addition to river embankment expansion.

<u>Removing existing river embankments</u>

If removing an existing river embankment to construct a sluiceway, the existing river embankments should be excavated wider so that structures and approach embankments can be readily constructed. On the other hand, in terms of river control, existing stable river embankments should be excavated as little as possible, and care needs to be taken so that the existing river embankment is not excavated more than necessary. The soil used for existing river embankments should also be carefully considered for slope gradient during excavation to avoid cracks on the existing river bank.

Chapter 4 Protection Works for river embankment (500 Protective Works)

Section 4.1 Toe protection works

4.1.1 Cement concrete (CC) blocks

River embankment slopes may be damaged or scoured due to water flow, bed scour and waves. To prevent damage and scouring, the slope can be covered and protected by CC blocks.

Block dimensions and manufacturing quality

The standard CC block dimensions, mixture and strength at the time of manufacturing can be categorized in the following table.

Table 4.1.1 (1) CC block quality by category

Quality			Reference No.
28 days compression strength (N/mm^2)	Leanest mix (cement:sand: stone)	Classification of stone (course aggregate)	
9.0 N/mm^2	1:3:6	Stone Chips (40 mm downgraded)	40-140
		Shingles (single particles) (40 mm downgraded)	40-190
10.5 N/mm^2	1:3:5.5	Stone Chips (40 mm downgraded)	40-200
		Shingles (single particles) (40 mm downgraded)	40-210

Table 4.1.1 (2) CC block standard dimensions

Ref.No. 40-160/ 210-	Dimension (cm)		V (m3)	Ref.No. 40-160/210-	Dimension (cm)		V (m3)
	W × L ×	H			W × L ×	H	
10	60 × 60 ×	60	0.216	55	30 × 30 ×	15	0.0135
15	50 × 50 ×	50	0.125	60	25 × 25 ×	25	0.0156
20	50 × 50 ×	40	0.100	65	20 × 20 ×	20	0.00800
25	45 × 45 ×	45	0.0911	70	45 × 45 ×	20	0.0405
30	45 × 45 ×	30	0.0607	75	40 × 40 ×	15	0.0240
35	40 × 40 ×	40	0.0640	80	50 × 50 ×	60	0.150
40	40 × 40 ×	20	0.0320	85	50 × 50 ×	25	0.0625
45	35 × 35 ×	35	0.0428	90	50 × 50 ×	20	0.0500
50	30 × 30 ×	30	0.0270	95	40 × 40 ×	15	0.0240

Note: Reference No. refers to the items in the Standard Schedule of Rates Manual (SSoRM) from BWDB

Notes when manufacturing and installing CC blocks

a)Use sand with a fineness modulus (FM) of 1.5 or higher.

b) Clean gravel to be used with water to remove impurities.

c) Compact sufficient raw concrete in molds and cure for at least 21 days.

d) Sufficiently compact the area where the blocks are to be laid and remove any surface water before laying them.

e) Casting method of concrete and block laying manner are carried out upon instructions from or approved by the Engineer in charge.

4.1.2 Guny bags (refer to Item Code: 40-400 ～ 40-470)

1) Type and size of gunny bags

For bank protection, sand bags are caterglize various types acoording to mareial of bag its, filled material and size as follows;

Table 4.1.2 Types of Sand bags according to the material

Passes / Material Of bag itself		Filled material in the bags				
		Sand or earth	Sand or earth Available at site	Sand of FM≧0.80	Sand cement; Sand : cement 10:1	8:1
New gunny bag:	40-400	40-420	40-440	40-450	40-460	40-470
2nd hand gunny bag:	40-410	40-420	40-440	40-450	40-460	40-470
Synthetic bags:	40-415	40-420	40-440	40-450	40-460	40-470

Figures: refer to ItemCode of Standard Schedule of Rates Manual of BWDB

Size and volume of bags are as below;

Capcity 50kg: filled volume = 0.030 m^3

Capcity 75kg: filled volume = 0.040 m^3

Capcity 50kg with sand cement: filled volume = 0.026 m^3

Capcity 75kg with sand cement: filled volume = 0.031 m^3

Note: Volume of bags filled with sand cemet volume is 3/4 of normal ones

2) Sand-Cement Gunny bags

(1) Normal usage for slope protection

Snad-cement gunny bag are usually used on slope of the river embankment for slope protection. Typical slope protection is shown following phtpo;

Photo 4.1.1 With sand-cement gunny bags Slope Protection on the left bank of Manu River, Moulvibazar

(2) Special usage for the foundation of river bank embankment

Sand-cement gunny bags are also used for the foundation of river bank embankment, making the shape of bsement part of river embankment in order to strengthen to the bottom part of embankment. Following is one of example of such case showing its cross section;

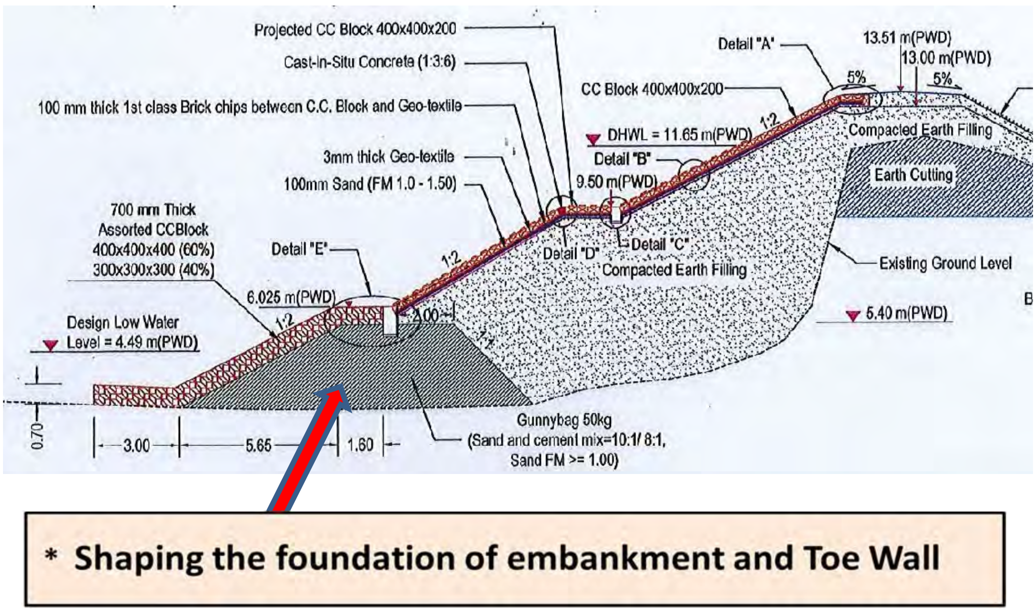


Figure 4.1.1 One of cross-section of sand-cement gunny bags foundation

(3) Example construction of sand-cement gunny bags for embankment foundation

A. Traial Fabrication of sand-cement gunny bags

1. Objective of Trial Fabrication for sand-cement gunny bag (S-C GB)

In order to define water cement ratio & curing days in fabrication of sand-cement gunny bags, Trial Fabrication will be carried out on different test items (factors), such as i)proportion of mix (sand : cement), proposed sand and curing days. Furthermore, workability and strength for the foundation of toe of embankment may also be rectified.

2. Fabrication of S-C GB on 2nd December, 2015

According Plan of Trial Fabrication, 11 different mixture and total 99 nos. of S-C GB have been fabricated. Type and mixtures of trail fabrication are as below;

Mixture SI No.	Sand Cement ratio	Sand type	kg as per half pack of cement			W/C
			Water	Cement	Sand	
M-1	10:1	Medium	20.25	25.0	250.0	0.810
M-2			23.35	25.0	250.0	0.933
M-3			26.60	25.0	250.0	1.064
M-4	8:1	Medium	19.05	25.0	200.0	0.761
M-5			21.70	25.0	200.0	0.867
M-6			24.50	25.0	200.0	0.979
M-7	6:1	Medium	14.70	25.0	150.0	0.589
M-8			16.75	25.0	150.0	0.671
M-9			18.95	25.0	150.0	0.757
M-10	8:1	Local	24.50	25.0	200.0	0.979
M-11	8:1	Coarse	24.50	25.0	200.0	0.979

Checking workmanship after mixing sand cement

Measuring weight/volume of material

3. Inspection Sand-Cement Gunny Bag

1) After 7 days curing on 9th December, 2015

Dropping from 1 m above the ground, and check the strength against the impact

Inspection was carried out Workability and Condition of hardening by 1) measuring unity weigh 2) dropping test 3) scratch surface and other observation

2) After 14 days curing on 16th December, 2015

Measuring weight and volume and find out unit weight

Interim evaluation after inspection for bags of 7 & 14 days curing:

1) Hardness of 8:1 mixture is harder than 10:1 mixture

2) Coarse sand used one is harder than fine sand used one

4) 8:1 mixture is good enough dumping and no need to 6:1 mixture.

5) 7 days curing is good enough curing time and it is desired furthertesting for 3 days or 5 days curing bag checking.

3) After 3 & 5 days curing on 19th December, 2015

checking hardness and degree of hydration of mortar by scratching the surface

It is clear that S-C GB after 3 days curing is going hydration and not yet stiff enough for dumping. SC-GB after 5 days curing show the hardness as same as 7 days curing one.

4) Additional Inspection for 10:1 mix sand-cement gunny bags

By conducting additional fabriaction and inspection for 10:1 mix S-C GB, those also can be used the same porpose of the works, provided that it is preferable for enough curing days of more than 10 days.

4. Outcome from the result of Trial

Accoding to the result of Trial Fabrication, following factors might be difined in this Works;

1) Water cement ratio

Sand : Cement	Unit Water		Remarks	Sand Type
8 : 1	220 Kg/M3		cemet:225kg/M3 sand:1,797kg/M3	Medium
material of mixing	Site proportion of mixture as per batch of Mixer (Kg)			W/C= 0.979
As per one butch (half bag of cemet)	W:Water 24.5 kg	C: Cement 25.0 kg	S: aggregate(sand) 200 kg	

* In case of using 10:1 mix S-C GB, water cement ratio is also decided accordingly.

2) Proposed curing days

Proposed curing days is decided 5 days.

3) Desired type of sand

Proposed sand-cement gunny bag had better to be used high qulaity sand, medium sand in the Trial Fabrication (F.M.1.49 > 1.0), instead of local sand (F.M.=0.94)

B. Report of river bank foundation construction using snad-cement gunny bags

This is a report for sand-cement gunny bag works on the Pilot Repair Works of Manu River Flood Control Embankment in Moulvibazar District (Nov. 2015 -)in the Project for Capacity Development of Management for Sustainable Water Related Infrastructure.

1. Fabrication

Measuring by steel measuring box (23.45kg/box): Transfer bamboo bucket and 8.5 times of bucket =200 kg sand

1) Check the quality of material

2) Keep the proportion of mixture: one butch = a half of cement

8:1 mix : W=24.5 kg C= 25.0 kg and Sand=200 kg

3) Adequate mixing

2. Curing and stockpile

From 14 Dec2015, Start stockpile

Marking each day end

Curing at Yard 2 on 21 Dec 2015

Curing at Yard 1 on 8 Jan 2016

3. Dumping


On 21 Dec 2015, duping S-C GB started from the down stream


On 8 Jan 2016, ditto from the down & middle stream, 2,390bags dumped

4. Setting up proposed shape and Measurement


Setting up the location of S-C GB mound for dumping on 21 Dec rgi


Set formation level and alignment & check remaining works on 16 Jan op6

Every day, numbers of fabrication & dumping are registered

5. Difficulties

1) Water level of the River

Every day checking water level, WL=4.95 m (PWD) on 8 Jan 2016

By open/close of the gate of upstream Manu barrage affect the water level on site. Some times suddenly rise over 60 cm within only one day.
So information of open/close the gate should be known to avoid the damage from the normal works procedure.

2) Delivery of bags

Initial stage, access is narrow and steep for delivery

- Distance from stock pile yard to dumping position is quite long
- However the weight of bag 50kg instead of 60kg at Trial Fabrication, it is quite heavy for manual delivery through limited access route.

3) Hike of market price of gunny bags

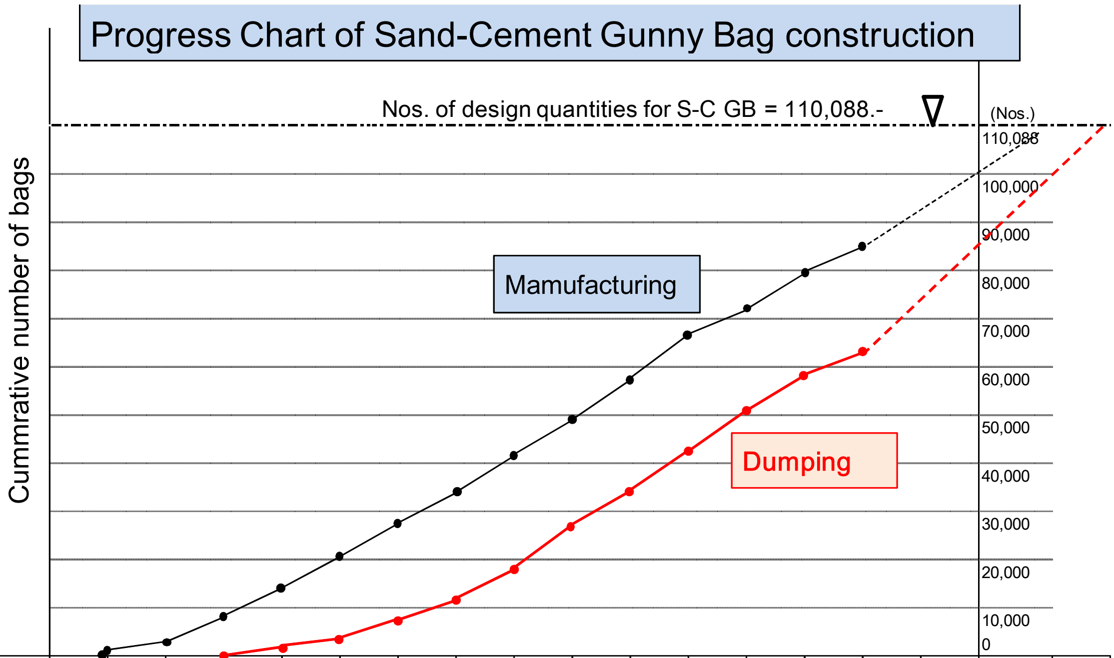
* Market price of gunny bag has been increasing recently
* Main reason of this hike is Government recent announcement that chemical fiber bag is prohibited to use for environmental affection

4) Checking shape of S-C GB mound under the water

- Final stage of dumping, S-C GB mound shall be checked as design shape.
- On land is Ok, but it is difficult to survey for the portion under the water.

* Data of construction

Progress of numbers of fbricated and dumped S-C GB is shown on the Chart below;



Date	14-Dec	17-Dec	20-Dec	23-Dec	26-Dec	29-Dec	1-Jan	4-Jan	7-Jan	10-Jan	13-Jan	16-Jan	19-Jan	22-Jan	25-Jan	28-Jan	31-Jan
Year			2015						2016								
Numbers of Fabrication (nos.)	335	3,168	8,700	14,691	20,110	27,450	34,495	41,962	49,156	57,401	66,138	72,698	79,622	84,628			
Numbers of Dumping (nos.)			0	1,817	3,865	7,235	11,691	19,237	26,156	34,748	43,238	50,425	58,173	64,023			

4.1.3 Geo Textile Bags

River embankment slopes, in particular the toe of the slope, may be scouring or erode due to water flow and waves. In order to prevent these scouring and erosion, geo textile bags can be used to protect the toe of the slope.

Standard specifications and dimensions of geo textile bags

The standard geo textile bag should be 400 grams per square meter in material mass, thicker than 3.0 millimeters, and should also be treated for protection from ultraviolet light. The standard geo textile bag dimensions are shown in the following table.

Table 4.1.3 Geo textile bag dimensions by category

Reference No.40-485/487/ 490/495-	Size of Geo textile bag		Weight (Kg) (filled with sand)
	W × L (mm)	V (m3)	
-10	1200 × 950	0.1664	250
-15	1100 × 850	0.1333	200
-20	1050 × 800	0.1164	175
-30	900 × 700	0.0840	125
-40	850 × 650	0.0730	110
-50	750 × 600	0.0520	80

Note: Reference No. refers to the items in the SSoRM from BWDB

Notes when sand packing and dumping geo textile bags

a) The FM of sand to be packed should be greater than 1.0 so it does not leak out of the textile mesh. FM larger than 1.2 is preferable.
b) After packing sand into bags, use a sewing device to firmly close the top of the bag so it does not leak sand.
c) When dumping geo textile bags onto the provisional falling apron, do so from a stable location, such as a fixed barge or flat pontoon. For eroded river embankment areas, unload the bags from a barge, pontoon or the deck of a boat. This may not apply in emergencies, such as disaster recovery.
d) Determine the bag dumping location based on the bathymetry survey result. Measure and record the loading location by a total station.
e) Follow geo bag procurement, on-site production, installation location and method on the technical specifications, approved drafts, regulation of tender documents, and instructions from the Engineer in charge.

(a) Sand packing

(b) Closing bags (sewing device)

Photo 4.1.1 On-site geo bag sand packing

(@ Town Protection; In Rajshahi COTE College (military preparatory school), around the landing place on 2013.10.8)

Section 4.2 Slope Protection

River embankment slopes may have surface damage or scouring due to water flow and waves. Rain may also wash away the slope surface. In order to prevent damage, scouring, and washing away the slope surface, various slope protections can be applied to the surface of the river embankment slope as follows:

4.2.1 Planting Grass

Plant is to be on the following specification as well as instructions from the engineer in charge.

1) <u>Sodding</u>

The following is the standard sodding work for the river embankment slope and crest:

a) Dense, well-grown sod should be used. It should be 75 millimeters thick, good quality durba and charkanta sods of size 20 cm square.
b) After sodding, compact and water the sod until it grows well.
c) Stake the sod appropriately and spread sand on the surface so that it does not slide off the slope.
d) Sod should be inspected for measurement after sod grows with root.

2) <u>Planting vetiver</u>

Standard planting procedure with vetiver to protect river embankment slopes from rain grooves, surface erosion, and waves should be as follows:

a) Use Vetiver or Binnah grass.
b) Plant at horizontal and vertical intervals of 20 cm.
c) After planting, maintain, compact, spread sand, and water the grass until it grows well.
d) Inspect sod that takes root and
e) grows normally.

Photo 4.2.1 Vetiver grass

4.2.2 Soil leakage prevention material (SSoRM: 40-600, geo textile fabric)

Use the soil leakage prevention materials as filter material show below to prevent soil from leaking due to residual water level and other reasons on embankment bodies that use permeable materials, such as structures in water, and where water touches the embankment body.

a) Soil leakage prevention filter fabric material should be anchored at a depth from the embankment body surface which ensures enough surface cover. The vertical height of such filter material should not be higher than the low water level and provide necessary lapping length.

b) The standard quality of the leak prevention geo textile fabric material is as follows:

Elongation: 60% ≤ MD ≤ 100% (maximum tensile direction)
40% ≤ CMD ≤ 100% (maximum tensile)

Permeability: $K \geq 2\times10^3$ (at $2kN/m^2$)

Proper joint: Mechanical continuous joint (polypropylene(PP) or nylon)
35 cm wrap width while drying or
100 cm wrap width in water

Ultraviolet rays (UV) protection: Protection from UV rays or other harmful compounds

ISO certification: ISO certification and manufacturer information should be shown on the geo textile material when delivered to the site.

c) The procurement, on-site production, installation location and method of geo textile for preventing soil leakage should be based on the technical specifications, approved drawings, provisions of tender documents, and instructions from the Engineer in charge.

4.2.3 Clay Blanket

<u>Covering work (river embankment surface) using cohesive soil</u>

When making an embankment body with sandy soil, there are times when cohesive soil (clay) is used to protect the embankment surface to prevent the sand from being washed away by an increase in river water level or rain. The thickness of clay blanket depends on the materials to be used and river conditions, but is generally 0. 5 to 1.0 m.

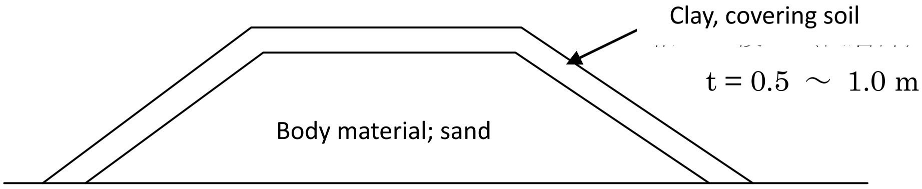


Figure 4.2.1 Covering the surface with clay

General standards and specification of construction of clay blankets are as follows;

a) material; Proposed desired clayey material composes mainly of clay and some of silt. Composition of specified soil must be confirmed by laboratory test.
b) Layers and ranges; Throwing clay for one (1) layer is not exceeding 150mm in thickness with clod breaking and benching the side slopes.
c) Compaction; The compaction degree is to be 85% to 90% of maximum dray density at optimum moisture content defined by the specification.

4.2.4 CC-blocks covering

CC-blocks are utilized as slope protection of river embankment with covering the surface of banks as well as usage for toe protection described in Clause 4.1.1 Cement concrete (CC) blocks avobe. Fabrication of CC-block for slope protection is the same as the manner of afore said CC-block for teo protection in Clause 4.1.1.

CC-block covering is to be constructed with a) erosion protective material and b) filter material underneath of CC-block layers as below;

1) Erosion protective and filler material

a) Geotextile fabric material (40-600)

As aforementioned in clause 4.2.2 “Soil leakage prevention material”, Geotexitile fabric material is used in order to prevent soil leakage from the river bank underneath of CC-block covering on slope of river embankment.

b) Filter material for CC-block covering

As a basement and filter for CC-block covering on bank slope, following materials are to be laid underneath of such CC-blocks;

i) broken brick chips (refer to ItemCode of SSoRM: 40-610)
ii) shingles and peagravels (diito but 40-620)
iii) stone chips (diito but 40-640)
iv) sand (ditto but 40-650)

2) Prevention measures for reduce the flow velocity

In order to reduce the velocity of river water flow, various measurement or methods have been proposed. Some of them are introduced here as below;

a) Alternately allocation

Different size or height of CC-blocks is laid on the slope of river embankment or on the slope of revetment of bank, alternately in position of the slope in order to decrease the flow velocity. One of sample photo is as left in the seashore inner part of curving.

Photo 4.2.1 CC-block on Revetment

b) Block with projection part

There is a CC-block with projection part which ordinary had been previously fabricated in the factory. In case of casting the concrete on site, most attention has to be taken regarding setting formwork and quality of concrete. One example of this type of block is shown on the photo below.

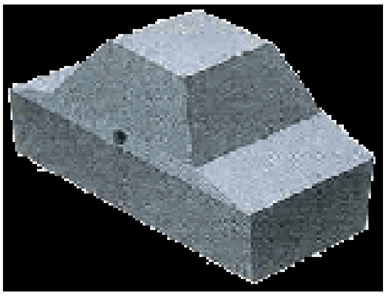


Photo 4.2.2 Example of prefabricated CC-block in Factory and layout on the slope

(Example size: 480mm×250mm×250mm (45.7kg))

Example construction of Slope Protection using CC-block with projection part

<u>Traial Fabrication of CC-block with projection part in the Pilot Repair Works</u>

（Quated from “Report for Trial Fabrication for CC-Block 40*40*20cm with projection part, Supervising Team / JET and BWDB Moulvibazar O & M Division on March 2016 and others）

Name of Works: The Pilot Repair Works of Manu River Flood Control Embankment in Moulvibazar District In order to make sure the productivity, workability and practicality of CC-block 40×40×20cm with projection part when manufacturing, Trial Fabrication for the block has been conducted. For reference of actual works, the results, findings and recommendations from the Trial are reported herewith.

1. Objective and characteristic of CC-block 40×40×20cm with projection part:

Introduction of CC block with projection in order to decelerate flow along the protection work and to limit generation of secondary flow along the protection work has been adopted in the Pilot Repair Works.

It is challenging action for improvement of designing and construction works for river embankment/ revetment under the Capacity Development Project.

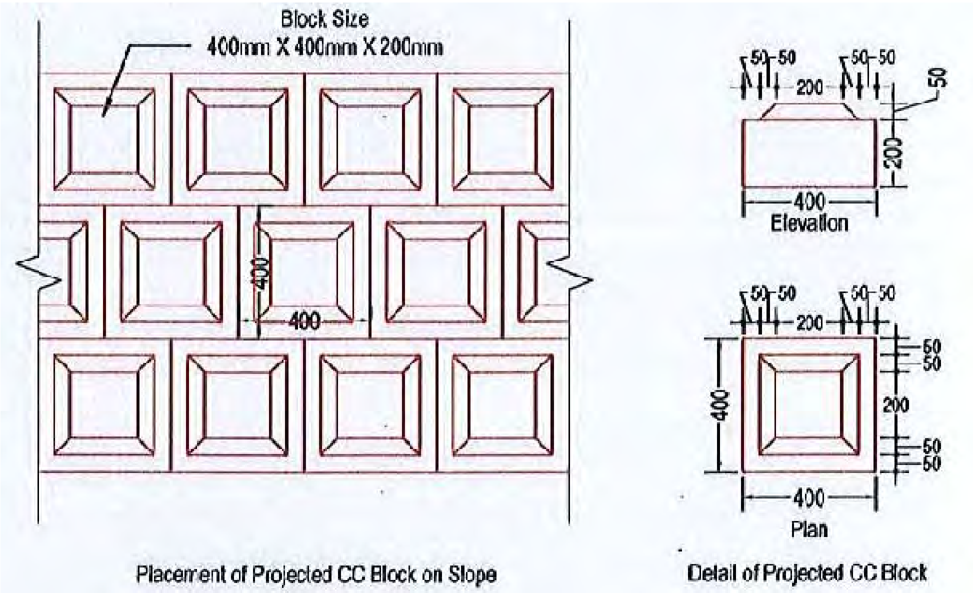


Figure 1. CC-block 40×40×20cm with projection part for Slope Protection:

2. Checking points of Trail Fabrication:

Checking points of the Trial are followings;

1) Formwork for projection part
2) procedure of casting concrete for projection part
3) workmanship of manufacturing and inspection of completed shape of the block
4) difference between the concrete mixtures

3. Results and Findings from Trail Fabrication:

From 2nd to 7th March 2016, Trail Fabrication for CC-block 40×40×20cm with projection part has been conducted. Observation of casting concrete and inspection for Formwork and workmanship of completed shape of the blocks and others are recorded as below tables;

Table. Checking results for Trail Fabrication for CC-block 40×40×20cm with projection part

Checking Point / Date	Formwork	Casting concrete	Workmanship and shape of the block	Concrete mixtures (1:2: 4 mix/ 1:3:6 mix)
02 – 07 March 2016	1) Number of formworks are need more 2) Checking dimen-sions of the Form 3) Stiffness of Formwork	1) Setting Formwork 2) Scratching the surface of lower part 3) Cement grouting on the surface of lower part	1) After de-mould, it looks not bad with some touching -up plastering 2) Dimensions of the completed shape of CC-block with projection	1) There are no difference between 1:2:4 and 1:3:6 mix concrete when casting 2) No difference found regarding the shaping the block
Result of Checking	Good	Not bad	Good	No difference

4. Recommendation for manufacturing CC-block 40×40×20cm with projection part:

After above checking result and taking consideration for the productivity, workability and practicality of CC-block 40×40×20cm with projection part when manufacturing, following recommendation are made with some comments;

1) Procedure and method of manufacturing, carried out at Trial Fabrication, is satisfactory for the manufacturing for CC-block 40×40×20cm with projection part for the Works
2) In order to catch up the Slope Protection Schedule, more steel made Formworks will be required if necessary.
3) It would be considered the concrete mixture for CC-block with projection part using not only 1:2:4 mix but also 1:3:6 mix
4) Since additional trials of fabrication, it is preferable that a holes on to the lower part of concrete (first casting concrete part) is to be made and the second concrete for the upper part (shaping special steel form for the projection part) is to be casted. The connection between lower and upper part of this method is more strong than that of only mortaring after scratching the top surface of lower part.

Attachment: Reference photos of Trial Fabrication for CC-block 40×40×20cm with projection part

Reference photos of Trial Fabrication;	
	
PhA. Checking steel made formwork, with require dimensions	PhB. Scratching the surface and cement mortar plastering before casting projection part
	
PhC. Checking the procedure of casting concrete and workmanship of manufacturing for projection part	PhD. Checking completed shape of projection part as per design
	
PhE. Ditto of Ph.D	PhF. Doing room for improvement increasing adhesion between lower/upper part, make a hole at the lower part

Chapter 5 Safety

The party in charge of the construction of river embankment should take great responsibility for the safety during such construction. To keep safety, It is necessary to provide adqueate safety facilities, to conduct safety management preventing construction workers and third party people from accidents/dessastors and to prepare the special protection measures when flood occurs in the river construction site.

Section 5.1 Safety Prevention Facilities

The safety facilities required for performing the work must be properly planned according to the stipulations in relevant regulations, adjusting to the work site and the surrounding environment.

(1) Signboards and observers

The signboards required for safety management must be determined before the work commences and posted in their correct positions. Watchmen against third parties should also be considered for safety as necessary.

Sufficient measures and care are required to prevent damaging the areas around existing river embankments in particular, which will often have buried gas lines, water and communication lines as well as housing nearby.

(2) Protective fencing

Areas considered dangerous during construction must be sectioned off with protective fencing and other hazard preventions, along with no trespassing signs.

(3) Dust prevention

In locations where dust may be generated from construction, dust preventive measures must be taken. Standard dust prevention measures include spreading water or asphalt emulsions.

(4) Water pollution control

If the construction may pollute the water, measures such as providing fences for water pollution fence, a sedimentation facility or chemical treatment are to be considered.

Section 5.2 Safety Management

Construction sites often change from hour to hour due to the difficulties in achieving a simple and uniformed working conditions. Therefore, the management system has to be able to respond to these changes.

The fundamental principle of safety management is safety first, and pursuing this principle leads to the speedy and economical creation of good work. Attention must be paid to safety from the planning stage to the construction stage. Care also needs to be taken to ensure that the on-site construction system and safety measures are not detached from each other.

Furthermore, extra care is required when reinforcing or expanding existing river embankments in comparison to the construction of new river embankments as it is more likely that accidents happen to third parties with river embankment crests and berms acting as combined roads or being adjacent to cities.

5.2.1 Regulations and checkpoints in regards to safety management

Regulations that are concerned with safety management are The Bangladesh Labor Act (2006) and the audit on occupational health and safety (implementation of safe construction) and are enforced by the Chief Inspector of Factory and Establishment of the Ministry of Labour and Employment on construction companies.

The following are checkpoints for safety management:

a) Establish health and safety committees
b) Establish a plan for important construction, equipment, and machinery (excavation of land that is higher than 10 m, temporary roads, etc.)
c) Work that should be carried out by work supervisors and operation chiefs (excavation of land that is higher than 2 m, earth retaining support, construction machinery repair and removing attachments, etc.)
d) Qualified workers (blasting engineers, construction machinery operators, etc.)
e) Work that requires the allocation of watchman and flag man (prevent machinery from falling and colliding, work close to overhead power lines, open-cut excavation work, etc.)
f) Set up no entry areas (within the work radius of machinery, land that is prone to collapsing, locations with hazardous materials, etc.)
g) Presentation (work plans, signs and warnings)
h) Compulsory wearing of protective equipment (helmet, safety belts, monitoring whether equipment is being worn, etc.)
i) Other work orders to be complied with against prohibited items (excavated surface gradient, earth retaining structure, machinery speed limit, prevention of electric shocks, etc.)

5.2.2 Serious accident and Cause against construction disaster

1) Three biggest Serious Accidents

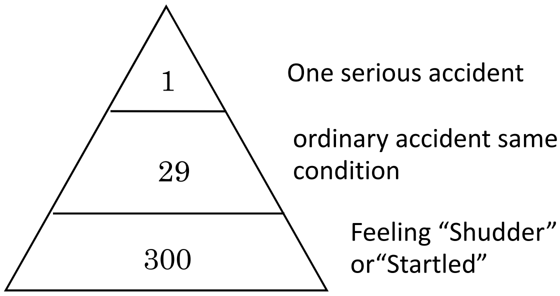
In the construction site, there are three (3) biggest Serious Accidents; falling and dropping, accidents related machinery and cpllapse of earth. The following shows the main construction disasters:

a) Accidents by construction machinery
b) Accidents caused by landslides or collapse during excavation
c) Falls from high places or falling objects
d) Traffic accidents within the construction site

When construction has to carry out close to the resident people such as re-sectioning and/or repair works for existing river embankment, it is likely to occur the third party accident especially traffic accident concerning to the third party.

2) Princple of Cause of Accidents (Heinrich's law)

There is one of the principle analyzed for construction accident experimentally which consists of one serious accident (with dead victim) accompied with 29 numbers of ordinary accidents and one ordinary accitent induced by 10 numbers of the phenomenon where workers feel danger or fear saying "Shudder" or"Startled"



It is imporatnt to prevent the accident on the sites that all the people on the site must not forget such phenomenon of "Shudder" or"Startled" and take necessary measure do not happen such suitation.

5.2.3 Preventing accidents on construction site

The following will be considered as precautions to prevent industrial accidents.

(1) Organize a safety management committee to plan and implement safety management, and promote systematic safety management.
(2) The Employer (the Contractors in the case of contract work) is encouraged to educate new workers when employing workers.
Education content includes:
  a. Construction General, management systems, organization
  b. Site rules and environment, facilities
  c. Group work with other jobs

d. Comprehensive work schedule
e. Safety construction cycle
f. Dangers and hazards of machinery, construction materials, and places, and how to handle them
g. Performance and handling of safety and protective equipment
h. Work procedures
i. Inspection when starting work
j. Causes and prevention of diseases that may arise from the work
k. Maintaining tidiness and cleanliness
l. Emergency precautions and evacuation in case of an accident or disaster
m. Other necessary items for work health and safety

The following need to be confirmed for the workers at the time of education:

・ Qualifications, experience
・ Emergency contacts, blood type and past illnesses
・ Medical history records

(3) Display a communication system for emergencies in a place where workers can easily see and familiarize themselves with it.
(4) Hold morning safety meetings, safety progress meetings, before work meetings (tool box meeting), and conduct safety education/drills and evacuation/firefighting drills to increase worker safety consciousness.
(5) Study the topography and geology of work locations thoroughly, and select machinery and determine construction work methods to plan an appropriate safety management plan.
(6) Assign a work leader to be in charge of important construction works. The work leader will be selected from the appropriate workers depending on the construction machinery or type of work.
(7) Wear protective equipment suited to the type of work, and use construction machinery that has the appropriate equipment installed and has passed appropriate inspections.
(8) Inspect protective equipment, electricity, electric tools, construction machinery, vehicles, ground, earth retention, and other necessary items before work. More over implement separate monthly inspections.
(9) Store oxygen, acetylene, and fuel separately. Make sure that storage locations have fire extinguishers and install leak prevention devices for fuel storage to protect the soil from contamination.
(10) Determine a site speed limit and clearly display and advertise it.
(11) Organize a route for construction machinery and allocate the appropriate flag man to

prevent machinery from tipping or colliding.

(12) When leaving machinery, stow buckets and earth removal plates, stop the engine, remove the key, and lock the door to prevent the machinery from rolling away or incorrect operation by workers other than the responsible operator.

(13) When using a mobile crane, extend the outriggers to their maximum position and place planks or iron plates underneath them. Preselect machines with appropriate performance according to cargo lift and work radius. Check the hook latch, over lifting prevention device, and limiter operation. Also check for kinks and breaks in both the main and slinging wires. Select a signaler who will check the cargo when it is being lifted and use supporting ropes if necessary without going under the cargo. When there is strong wind or thunder in the area, stop work, store the boom, and keep low. Also make sure the crane is grounded.

(14) In order to react to landslides and collapses during excavation, make sure to excavate with a safe gradient, and remove stones and soil that might fall from the slope. Prevent the erosion and collapse of the slope by placing a berm at the shoulder of slope. Also inspect the ground and pay extra attention to changes, such as cracks and swollen ground, after rain and monitor them if necessary.

(15) Erect a fence at dangerous locations to keep un-authorized persons out and raise awareness by setting up safety signboards.

(16) Place a handrail above cut slopes and at the shoulder of river embankments with a gradient sharper than 1:1.5 and a vertical height higher than 2.0 meters. If that is not possible, secure enough space and indicate the shoulder of river embankment.

(17) Ideally, berms should be set at 5.0-meter intervals for cut slopes and river embankments that exceed a vertical height of 5.0 meters.

Example of Safety Gethering and Saftey education to the workers

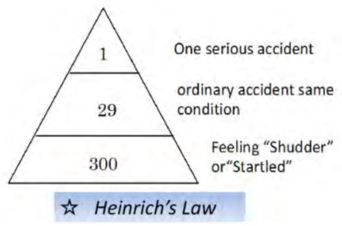
<u>Demonstlation of Safety Gathering held on the Pilot Repair Works</u>

Objective of "Safety Gathering":
To promote participants' awareness of Safety during construction works

Order of "Safety Gathering"

1) gathering everybody on site
2) providing safety goods and everybody wear it
3) safety address, such as 1:29:300 theory, by Engineer in charge of the Pilot Repair Works
< Education to all of the Workers>
4) "Safety First" Call in Bengali word ("Shabar Age Nirapatta") leaded by a representative of the workers

Reference:



Photos : "Safety Gathering" on Pilot Repair Works of Manu River Flood Control Embankment in Moulvibazar District held on 23rd January, 2016

Ph A. 150 workers and 40 WS participants gathered

Ph B. Safety Gather was conducted 9:05 – 9:25 am

Ph C. Safety idea is introduced (Lecture by PM)

Ph D. Emphasized "Safety First"

Ph E. Closing with safety first "Call"

5.2.4 Preventing accidents to third parties

The following will be considered as to prevent accidents to third party;

a) Select construction methods that account for prevention of disasters to the public.
b) Clearly indicate and distinguish areas that are necessary for construction by setting up fences and the like so that third parties do not mistakenly enter. In the event that someone does enter the site, give consideration to preventing accidents, such as falling into holes.
c) Consult with road administrators and the local police station if necessary to decide construction traffic routes. If roads are used, make sure the necessary procedures are carried out and precaution signs to inform the general public.
d) At construction vehicle entrances and exits, enforce drivers to stop once, and if necessary place entrance and exit signs and allocate a flagman.
e) To prevent dangerous traffic and traffic jams, clearly indicate the transportation route, place necessary direction signboards, and allocate traffic guard to ensure safe traffic.
f) If dump trucks travel on public roads, prohibit overload (allocate watchmen) and prevent dust (install a wheel washing system, remove and clean mud, enforce tailgate closure, cover the load with a sheet, etc.).
g) Conduct trial excavation for work adjacent to buried objects such as gas pipes, water pipes, and electric and communication cables. Confirm their plane position and depth and take necessary safety precautions with administrators in attendance.
h) Pay close attention to construction machinery usage and movements by allocating watch man. Also take appropriate action to prevent machinery from tipping due to soft ground (lay iron plates or timber mats, improve the foundation, etc.).
i) In order to prepare the case that an accident occurs from hitting buried objects or overhead electric wires or those objects are broken, clarify to contact person(nel) of utilities in advance and review emergency procedures.
j) Keep the construction site tidy and orderly so that dust from the construction does not disturb the surrounding areas.
k) Display a construction organization chart where third parties can visually check the construction system and the responsible engineers for each construction work.

Section 5.3 Flood Prevention

In principle, river training work should not be performed during flood season. When there is no other alternative, care must be taken with the method and order so that the setback does not collapse during the flood season. Care must especially be taken for earthworks close to the embankment body. It also is required to take necessary measure preventing flood disaster even in dry season.

5.3.1 Flood countermeasures

Create an emergency plan, procure staff and equipment, and c larify the division of roles and chain of command with an organizational chart.

In certain cases in river training work, unusual amounts of rainfall may result in flooding even in the non-flood season. As such, always keep the following in mind during the work period:

a) Gather information such as weather forecasts and river information.
b) Anticipate rises in river water levels by consulting records of past rainfall and water levels.
c) Inspect the work site to deal with flooding quickly and effectively.

The following measures are listed for flooding:

(1) Check work progress just before flooding

If flooding is expected, work progress must be checked just before flooding hits.

(2) Secure flood preventing equipment

If there is a chance of flooding during embankment or excavation work affecting the target structure, consider stocking drainage pumps, sandbags, waterproof sheets or other emergency treatment and recovery equipment in normal times to ensure flood preventing capacity.

(3) Sheltering for people and machinery in floods

It is important to decide a storage space for machinery and evacuation route and to hold evacuation training in order to safely and smoothly shelter workers and machines against unexpected or nighttime floods.

Also, a safe location to moor and other considerations will be needed for work ships in case of floods.

(4) Emergency organization at flood

The following items must be determined in advance for emergency flood situations and taught to all staff on site.

b) Staffing plan (organization chart)
c) Chain of communication (emergency contact sheet)

(5) Flood response

In the case of a flood, swift and precise action will be required. Set up an emergency organization immediately with gathering the information of rainwater and water level, inspecting the site and relaying necessary information to those in the predetermined chain of communication.

5.3.2 Preparing flood countermeasures during earthwork

When a flood is anticipated, prepare sandbags and other flood prevention materials and take necessary action so that river structures are not threatened due to construction. Even outside work hours, the mobilization system for staff and construction machinery must be ready.

5.3.3 Additional prevension measures during the suspention of the Works

When the Works cannot be continued or are to be suspendited during the monsoon and pre/post monsoon season, temporary prevension works may be carried out according to the site condition and the nature of the Works.

In case of additional temporary prevension works is planning, the necessity and the effects of such prevension works must be considered among the all parties concerns of the project/the Works. Following is one example of additional prevension measures conducting in one Pilot Works in river embankment.

“Temporary Prevention works during monsoon season”

(Order of Variation No.4)

Name of Project: Capacity Development of Management for Sustainable Water Related Infrastructure, BWDB / JICA

Name of Works: The Pilot Repair Works of Manu River Flood Control Embankment in Moulvibazar District

1. Reason of the suspemsion of the Works and the progress of the works

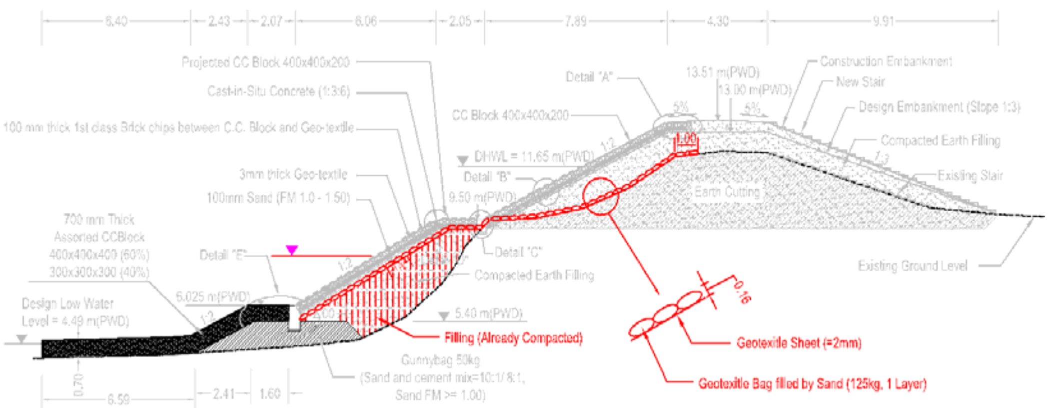
It has been extra ordinaly imclemnt weather in the construction works of the Works, such as extraordinary inclement weather on 25th February 2016 and heavy rain fall and its consequent water rise (caused by flash flood) from 28 March 2106. At the middle of May 2016, the water level has been above the bottom level of slope and it would be expected to be continute above such level. Under such situation, slope protection works cannot be continued. Thus the Works were suspendited during coming monsoon season until next dry season.

The main works done in the Contract by the middle of May are as below;

1) By the 29 March 2016, manufacturing CC-block (both 40:40*40cm and 30*30*30cm size) has completed and dumping of them has almost completed.
2) Earth work; Cutting & construction of embankment is completed about 90% and 80% respectively by the end of April, 2016
3) The numbers of Slope Protection CC-block has been manufactured by the 24 May 2016; 40*40*20cm with projection part and others have been almost completed.

2. Prevension works and method

1) As joint Inspection, BWDB Design Circle, BWDB Moulvibazar O&M, JET and TSS, the “Temporary Prevention works” (TPW) has been proposed and planned to avoid the further damage during coming monsoon season, and JICA approved it as Variation No.4 to carry out the works.
2) TPW includes laying geo-sheet to the executing and under-constructing slope and placing/dumping geo-bag with sand (125kg) to prevent the slope from washing away with executed embankment material by flow/movement river water during monsoon season (including pre- and post- seasons) and minimize the damage for re-starting Pilot Repair Works.
3) Proposed drawing of standard section for TPW;



3. Reference photos of prevension works for during monsoon season

Dressing the surface of earth of slope before laying Geo-sheet on 2016.06.2

Laying geo-sheet and geo-bag on the slope on 2016.06.02

Ditto but on 2016.06.06 :
From down-stream to up-stream

Condition of the slope during monsoon season @2016.08.23

Chapter 6 Inspection

There are two inspection, i.e. one is construction inspection conducted during the execution of the construction and the other is completion inspection when the Works has been completed to confirm the its completion.

Section 6.1 Inspection for Construction

6.1.1 Objective and Notes of Inspection

1) Inspection objective

Inspections are meant to check whether the construction is being implemented as contracted upon completion and partial completion by observing, measuring, testing, and other means, and to determine whether the work is acceptable. The main objective of inspections is to guarantee the finished form and quality of the construction work based on the Contract.

2) Notes on river embankment inspections

The quality of river structures requires uniformity. It is important not to miss the big picture by focusing too much on the details. It is therefore unforgiveable to consider the completed form uniformly when some parts are too large but others too small to maintain balance.

The inspection is to check whether construction was implemented based on the design, specifications, drawings and other contract documents, and whether the target structures were built.

Items for inspection are as follows;

Table 6.1.1 Items for Inspections

Inspections items / Control items	work item / Items for Inspections	
Quality	Material	Record of manufacturing, Test record at delivery
	Earth work	Grading of fill material, Degree of compaction, Construction water content
	Protection	Compressive strength of concrete
Measurement of works	Length of river embankment, Elevation and width of crest, Grade of slope	
Construction Progress	Commencement date, Completion period	

6.1.2 Inspection types

1) Daily inspections

In principle, measurement of works and quality are to be inspected daily. In particular, items where quality and the completed form cannot be checked afterwards due to further work must be inspected at each construction steps.

The following are concrete examples:

a) Measuring the water content of fill material that will be compacted during embankment construction.
b) Spread thickness and degree of compaction for each layer during embankment or backfilling work.
c) Inspection of rebar arrangement status for reinforced concrete structures.

2) Phased inspections

Phased inspections check the completed form and the works done at each phase of the work (indicating work progress by the cumulative construction amount). There are inspections for each phase to check construction progress at the end of each month or for cases with regulated partial payments.

Quality inspections at important work and type phases are another type of phase inspection. Phased inspections can be used in the following situations:

a) Current ground elevation and shape of excavation or embankment before starting the work.
b) Quality inspection for whether the fill material has certain characteristics before embanking work.
c) Quality inspection of cement and rebar to be used to manufacture concrete blocks.

Furthermore, phased inspection can be the foundation for partial payment. In this case, the completed form is inspected where mainly quality has been inspected satisfactory and work progress that shows the cumulative total amount can be calculated based on the cumulative total quantity.

Section 6.2 Completion Inspection

6.2.1 Objective of Completion Inspection

The completion inspection is conducted for confirmation of the completion of the contracted work, hand over the permanent structure to the client and determines the payment to the contractor upon completion, the completion date of construction, and the start date to repair any defects. Once the work has been confirmed to meet the purpose defined in the Contract, a certificate of completion is issued. As the certificate of completion shows the rights and obligations of the parties concerned upon completion, a completion inspection is important to complete the work.

6.2.2 Notes of Completion Inspection

1) Role of inspectors

The completion inspection is conducted by the client, so inspectors who manage the inspection, determine whether construction has passed or failed, and confirm that the inspection has been completed by persons put in charge or delegated by the client.

2) Order of general inspection

While the order of inspection depends upon the type of work, inspectors and the situation at the time of the inspection, the general flow is as shown in Figure 6.2.1.

3) Required documents at the time of the completion inspection

The following deliverables need to be prepared for the completion inspection.

a) Completion documents
b) Quality control records
c) Construction log photos
d) Calculation documents of quantity for completed form (calculation sheet for quantities)
e) Completion inspection log

4) Checking repaired works

When items are pointed out for repairs during the completion inspection, they need to be repaired. After patching work is finished, these repairs will be checked by the inspector or parties delegated by the inspector. The inspector will then sign the inspection log to complete the inspection.

5) Certificate of completion and hand over

After confirming that the completion inspection has been conducted, the Client issues a certificate of completion to the Contractor and the objective of the Contract is handed over to the Client. The defects liability period needs to be stated in the contract document upon completion and handed over.

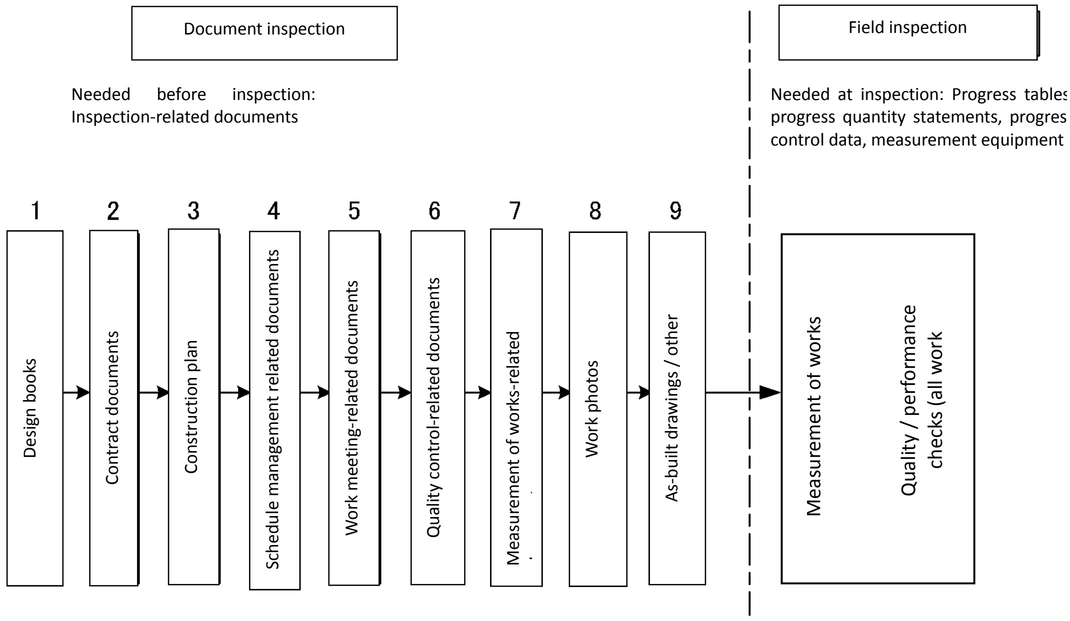


Figure 6.2.1 Order of general completion inspection