5. PROPOSED FCD PLAN AND PRELIMINARY DESIGN

5.1 Proposed FCD Plan for the Whole GIP Area

The FCD plan for the GIP area is composed of construction of flood embankment, river training works, channel excavation, compartmentalisation of the internal drainage area and construction of drainage structures. These components are aligned to be the optimum layout in view of their function, structural dimensions and costs in order to achieve the objective of FCD plan. The layout plan of the components for the selected FCD option in the previous chapter is described below and illustrated in Figures 5.1 to 5.3.

(1) Flood embankment

Flood embankment are planned for the right bank of the Teesta river and the left and right banks of the Ghagot river. The total bank stretches are 46.6 km and 108.6 km for the Teesta and the Ghagot rivers respectively. Its construction is divided into three work items, that is, provision of new flood embankment, resectioning/heightening of existing flood embankment for strengthening and resectioning/heightening of existing road embankment for utilizing it as flood emabnkment. The embankment works contemplated by each item is broken down as given below;

Item	Teesta	Ghagot left bank	Ghagot right bank
1) Provision of new flood embankment	13.2 km	15.7 km	26.3 km
2) Resectioning/heightening of existing flood embankment	26.7 km	44.2 km	6.4 km
3) Resectioning/heightening of existing road embankment	6.7 km	16.0 km	0.0 km
Total	46.6 km	75.9 km	32.7 km

Of the embankments along the Ghagot, the stretch of 32.7 km from the river mouth to Sadullapur functions as a backwater levee to cope with the backwater from the Brahmaputra river.

(2) River training works

Revetment and groyne are planned to be provided as river training works at the river stretches where severe bank erosion is taking place due to flood flow. These works are planned for the Teesta river only from the study result on river condition such as flood discharge, velocity, sedimentation and existing bank damage.

The contemplated river training works layout are shown below;

Item	Location to of River Training Works Planned for TRE				
	Outfall	Belka	Sundarganj	Tambulpur	Painalghat
1. Length of river reach	3.2 km	2.0 km	1.5 km	0.7 km	3.2 km
2. Revetment	3.2 km	-	0.3 km	0.7 km	-
3. Groyne(crest length)					
a) Reconstruction or rehabilitation of existing groyne	-	150 m (1 no) 250 m (1 no)	-	-	-
b) Construction of new groyne	-	250 m (1 no) 300 m (1 no) 100 m (1 no)	100 m (6 nos) 50 m (2 nos)	-	220 m (1 nos) 100 m (12 nos)

(3) Channel excavation

As clarified through the hydraulic modelling analysis as well as public participation, the insufficient drainage capacity of existing Manas regulator is one of the causes for the drainage congestion in the GIP area. Although it is sure that the Manas regulator is washed away, for the time being it is difficult to forecast when it is lost.

In the present study, therefore, we have proposed to build a shortcut channel to flow down the flood discharge of the Ghagot with 1 in 20 years return period. The new shortcut channel is aligned to branch off the Ghagot 0.6 km downstream of the confluence with the Manas. The new shortcut channel has a length of 1.2 km brom the branching point to the Brahmaputra. The channel is designed to have a constant trapezoidal section with a bed width of 40 m and side slopes of 1:2.

Also, a new shortcut channel is proposed at a suburb of Gaibandha town to protect the town area of Gaibandha from the flood of the Ghagot and the bank erosion due to the meandering. The shortcut channel for protecting Gaibandha town is planned at the site around 0.5 km distant from the northernmost area of the town. It has a stretch length of 0.5 km and trapezoidal section with a bed width of 30 m and side slopes of 1:2.

(4) Compartmentalization

The GIP area is divided into 31 sub-basins by the existing/planned flood and road embankments for the purpose of compartmentalisation. The compartmentalisation in principle aims to control the concentration of rainwater in order to alleviate the severe inundation damage in low lands. It is planned to fully utilize the existing road embankments except 6.3 km long compartment boundary in total length along which there is no embankment. At present, there are 7 such structures with opening in those embankments as bridge and culvert which have to be closed. On the other hand, 5 new sluiceways with control gate are planned to be provided to secure the waterway for the water supply during the dry season.

(5) Construction of drainage structure

Regulator and sluiceway are planned to be provided on flood embankment to drain out internal water to outer river. One regulator is planned at the most downstream portion of each sub-basin and sluiceways are planned to be built together with side drains aligned in parallel with the flood embankment so as to drain out locally standing water to outer river. The natural drainage channels in the GIP area are blocked up in many places as a result of improper provision of rural road embankment without any

drainage structures. To restore the natural drainage function, drain pipes are planned to be installed in the rural road embankment at 450 points in the project area. The regulator and sluiceway planned to be on the embankment is summarized as follows:

Work Items	TRE	GLE		BRE
		Left Bank	Right Bank	
1. Provision of new regulator	3 nos.	8 nos.	1 no.	1 no.
2. Provision of additional regulator	1 no.	4 nos.	-	-
3. Rehabilitation of existing regulator	5 nos.	-	-	-
4. Sluice way	5 nos.	13 nos.	11 nos.	-

5.2 Design and Planning Criteria on Major Structures

5.2.1 Design concept

The design criteria is established primarily for the purpose of selection and design of the optimum type of structures as well as quantification for the construction cost estimate at the feasibility study level. The selection and design of the FCD and related structures are made to allow the minimum construction and O&M costs and such disbenefits to minimize as land acquisition and resettlement costs in line with the policy of the FAP.

To attain the aforesaid objective, the followings are taken into consideration in establishing design criteria for the planned design works:

a) a use of local labour and construction material,
b) construction method,
c) minimizing operation and maintenance works, and
d) design which takes into consideration the cost effective.

5.2.2 Project components to be designed

The project works consist mainly of rehabilitation of TRE and rehabilitation/extension of Ghagot Left Embankment (GLE), river training works against the bank erosion of Teesta and drainage improvement in the GIP area. The major project components are as follows :

a) Resectioning/heightening of existing flood embankment
b) Provision of new flood embankment to seal TRE and to prevent influx of flood in the Ghagot
c) Rehabilitation of existing groyne
d) Construction of short-cut channel near Gaibandha town
e) Rehabilitation of existing regulator
f) Construction of new groyne
g) Construction of new regulator
h) Construction of embankment for compartmentalization
i) Construction of sluiceway and drain ditch
j) Construction of new road bridge on short-cut channel

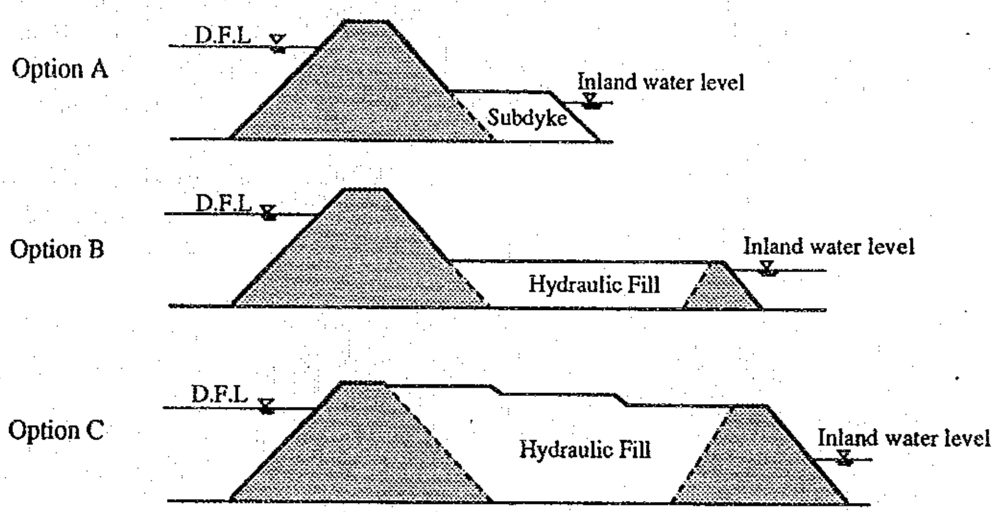
5.2.3 Design and planning criteria

Design criteria are prepared for required structures for FCD plan described in the Section 5.1 considering the following factors:

- local topography
- hydrological and hydraulic conditions in the GIP area
- geotechnical data available on subsurface soil and foundation conditions at structure sites
- availability of construction materials
- sociological and environmental impact

(1) Flood embankment

Inhabitants living along TRE have been using crest or slope of the flood embankment for building their houses by excavating the embankment body since their houses in the river side area were not compensated in construction of flood embankment. From the aspect of stability of embankment body in long-term period, these activities will not be allowed. To improve the mentioned situation, it will be required to resettle the above people in constructing flood embankment. The following types of flood embankment are conceivable as a multiple use of embankment for housing and cultivation:



A : flood embankment with a berm of a significant width behind the embankment; a width of 5 m is applied by FAP-8A, which is wide enough to use for housing.

B : flood embankment with wider platform at some locations to be constructed by hydraulic filling, which it is possible to use for cultivation as well as housing.

C : "superdyke" of hydraulic filling with wide crest width to provide same advantages as type B.

The above Options A to C are predicted to have difficulty for land acquisition according to our public participation discussing with inhabitants and officials concerned. On the other hand, the Options B and C also have disadvantage of higher construction unit rate than that of option A because of utilization of hydraulic filling with filtering and coffering works. In our preliminary construction cost estimate, unit rates for hydraulic filling by dredger and embanking by manual construction method are estimated at TK 250/m^3 and TK 69/m^3, respectively.

The Option A is judged to be a preferable option considering the least land acquisition area and construction cost among the Options. In addition, 5 m width of the land in the country side of the flood embankment will be needed as an access road or temporary working space for the construction works at least and this acquired land is useful for construction of subdyke for housing.

Typical cross sections of TRE and flood embankment along the Ghagot are decided based on seepage and stability analyses made on the both normal and seismic conditions. Dimensions of the typical cross section are shown in Figure 5.4 and described as follows:

a) Design flood water level along the river stretch is determined by the hydraulic modelling analysis using flood water level with 20 years return period and the river cross sections surveyed. The design inland water level is also determined for flood with 5 years return period.

b) Crest elevation of flood embankment is set by adding a free board of 0.91 m for GLE and 1.52 m for TRE and the backwater levee to the design flood water levels. Crest width is taken at 4.25 m following the existing design standard in this country.

c) Side slopes in both the river and country sides of flood embankment are designed at 1:2 for the Ghagot and at 1:3 for TRE in accordance with the design criteria adopted for the existing flood embankments. These slopes are reasonable from the aspects of bank stability and cost. However in case that seepage flow in the embankment produces an unfavourable effect on the bank stability, 2 m wide berm (berm-1) is provided on the slope of country side. Furthermore in case that the hydraulic condition around embankment is serious due to long duration of deep inundation, drain filter is provided at the toe portion of country side slope.

d) Basically, embankment material is obtained from the borrow area in the river side. Its distance from the toe of flood embankment is taken at 10 m (berm-3) taking into account slope stability. The excavation depth at the borrow area is limited to 2 m.

e) The minimum required safety factor of the slope stability of embankment is set at 1.25 under the normal loading condition and 1.10 under the extreme loading condition.

f) Stripping of surface soil, which includes organic matter and loosed surface soil is necessary for improving the bank stability against sliding.

g) Subdyke (berm-2) or land with 5 m width is provided on TRE to resettle landless people residing on the existing TRE. The subdyke is planned to be provided in case the gorund elevation at the toe of the flood embankment is lower than the internal water level with 1 in 5 years return priod. The latter is planned to be applied in case the ground elevation thereat is higher than the internal water level.

(2) Compartmentalization

The required height of embankment for compartmentalisation is determined through the hydraulic modelling analysis for GIP area adding free board of 0.5 m to the design inland water level. Basically, the existing road embankment is utilized for the purpose as it is if the crest level exceeds the required one. While, the existing road embankment needs to be heightened in case it does not have a sufficient height for the design internal water level. On the other hand, new embankments are provided where the drainage basin is newly compartmentalized into smaller ones. Side slope of 1 : 2 is applied to both sides of embankment. Crest width is taken at 3.6 m at least considering the function as a rural road. Existing openings in culverts and bridges are closed with earth filling. On the other hand, a sluiceway with control gates is provided instead of such opening at the site where a waterway is required for the water supply during the dry season.

(3) Groyne

There are two types of groyne whose use depends on the function expected, that is, one is permeable type and the other is impermeable one. The permeable groyne aims at reduction of flow velocity and promotion of sedimentation around the structure, whilst impermeable type is to change flow direction and maintain river banks against the direct attack of flood.

Wooden pile groynes of 5 to 10 m in length classified into permeable type is found along the upstream reach of the Teesta. However, the significant sedimentation effect is not identified thereat since its length is rather short and river bank erosion is mainly caused by existence of soft sandy soil layer along the river bank. Considering the present condition, it would be more practical to select the impermeable type of groyne which enables the present bank line to be maintained.

The impermeable groynes have been constructed along the Teesta with the same typical section as that of flood embankment. However, there are many cases that those groynes were washed away by the flood flow due to the poor construction material and method. To improve the present condition, it is required to give special consideration to the construction material and method considering the submergence of structure during monsoon season. The following function could be expected with proper arrangement of groyne group along a river stretch likely to be eroded.

- to control partial scour of river bed and unbalanced sedimentation
- to alleviate undulation of river bed along the longitudinal profile and cross section
- to create a floodway to allow smooth path of large scale flood

The basic dimensions of groyne are as follows:

a) length	:	10 % of river width during drought season
b) crest elevation	:	river bank elevation
c) crest width	:	5 m considering the submergence and the function as an inspection and maintenance road
d) spacing	:	10 times in height or 2.0 time in length.

(4) Revetment

The durable revetment will be provided at the sites where severe erosion is expected to take place as given below:

- slope and crest of groyne
- river bank at meandering portion
- bank slope at river structure site including up- and downstream portions of the structure

The following two types are conceived for the revetment work taking into account availability and reliability of construction material:

Type-A : river bank slope protection by brick mattress or concrete block casting with wire net and geotextile material such as jute sheet and foot protection by concrete block casting with wire net

Type-B : jute bag filled with river sand and wire net

Of the above, Type-B is much less costly but have disadvantage on resistance force against high flow velocity in the Teesta during peak monsoon season as compared with Type-A. Therefore, Type-A is adopted for the revetment works.

(5) Regulator

The planning criteria on drainage improvement of FAP 2 is to drain out the internal water to outer river by gravity flow through existing and/or new regulators within two (2) weeks immediately after the outer river water stage becomes lower than the water level at the outfall of drainage channel.

The flow capacity of regulator is designed on the basis of hydraulic model simulation. Most of the existing regulator in the GIP area is of concrete box culvert with small opening for gate control. A multiple gated type regulator is applied to the design and the gate will be manually handled.

(6) Sluiceway

Local internal water enclosed by road embankment or depressed lands is drained out through a sluice way. It is designed to be of in-situ box culvert with small flow capacity compared with that of the regulator. Since drainage area to be covered by one sluice is rather small (less than 2 km^2), dimensions of the structure are determined from the aspects of workability and maintenance works. A small steel flap gate is installed at the outlet portion and the internal water will be drained out in response to the difference of water level between outer river and internal area. Sluiceway may also be utilized as an intake structure for the water supply purpose during dry season.

Side ditch is planned inside and along the flood embankment. The water in local depressions will be collected by side ditch and drained out to the outer river through sluice way. Since the internal water is planned to be drained out by gravity flow, the alignment of side ditch is decided considering the local topography. It is designed to be of open excavation ditch with trapezoidal section.

(7) Drain pipe

Drain pipes are installed in order to lead rainfall water enclosed by the existing rural road embankment to drainage channels. Precast concrete pipe or corrugate pipe (to be selected from availability of these pipes in market) is adopted and the diameter is decided from maintenance aspect rather than flow capacity.

In consideration of drainage congestion due to siltation and/or garbage inside the pipe and the manual removal works, a pipe of 600-700 mm in diameter is installed. It is required to be covered with 0.5 m deep soil in the minimum.

(8) Road bridge

In the neighborhood of Gaibandha town, a new road bridge is planned to be constructed on a shortcut channel for the Ghagot which have a total channel width of about 100 m. Type of the bridge will be selected in consideration of the required total span length. The bridge width and the loading condition will be dependent on the road grade. The design criteria is summarized below:

a) Bridge type

Relation between bridge type and a span length is generally as follows:

Bridge type	Span length/1 span
pre-tension concrete girder bridge	less than 15 m
post-tension concrete girder bridge	less than 45 m
H-steel girder bridge	less than 20 m
reinforced concrete bridge	less than 10 m

In selecting bridge type, a total span length necessary and decrease of flow area due to bridge pier needs to be considered. In principle, the decrease of flow area has to be less than 5 % of the total flow area at the design high water level.

b) Bridge width

The required bridge width is dependent on road width. It is decided by adding extra space for a footway and a parapet to the road width. Since the existing road have a total width of about 6 m, bridge width is determined to secure this width.

c) Loading condition

The loading condition for bridge design follows the design standard of this country. In this design H-10 loading (loading condition for the second class bridge) is applied considering the present traffic conditions in the GIP area.

5.3 Preliminary Structural Analyses on Major FCD Structures

On the basis of the geotechnical investigation results, the preliminary structural analyses were carried out in terms of bearing capacity of regulator foundation as well as seepage, settlement and slope stability of flood embankments planned in the GIP area.

5.3.1 Structural and hydraulic analyses on planned embankment

(1) Basic design values and conditions

The preliminary structural and hydraulic analyses on the flood embankments were made concerning the seepage, settlement and slope stability of the typical cross sections. The basic design values required for these analyses, which consist of physical parameters, coefficient of permeability, consolidation and shear strength parameters were determined based on the geotechnical investigation results as summarized in Table 5.1 and Figure 5.5.

a) Typical cross sections of flood embankments

The following three typical cross sections of flood embankments are selected to be analyzed;

- Cross Section TRE-1 : New construction of TRE at the section where there are no embankments due to breaches and/or bank erosion

- Cross Section TRE-2 : Strengthening/heightening of existing TRE
- Cross Section GLE : Strengthening/heightening existing Ghagot left embankment

The above typical sections are shown in Figure 5.6. For these embankments, borrow areas are planned to be developed on the river side. The design high water of river and inland water levels and freeboard are determined in accordance with the design criteria thereon as shown in those Figures.

(2) Seepage analysis

The seepage analyses are carried out to obtain the seepage line in the embankment body and the time for the seepage to take to reach from the river side to embankment slope in the country side and to examine the possibility of piping failure in the embankment bodies.

a) Seepage line

The seepage line is analyzed for the following cases using the Casagrande's method explained in Figure 5.7;

case-1 : inland water level is equal to the ground surface
case-2 : inland water level corresponds to the design inland water level

During the monsoon season, there takes place the seepage through the embankment body and/or the foundation soil layer when there is a hydraulic difference between water levels in the river and country sides. In principle, typical cross section of the flood embankments needs to be designed so that the seepage crossing with the embankment slope surface in the country side be above the ground surface in order to avoid the slope failure due to the seepage breaking out of the embankment slope surface during the monsoon season.

The seepage line is estimated by initially drawing the line on the deformed section which is prepared by reducing the horizontal dimension by the ratio of kv (coefficient of permeability in vertical direction) to kh (coefficient of permeability in horizontal direction), and then by plotting the results on the original section. As a ratio of the vertical coefficient of permeability to the horizontal one (kv/kh), 1/10 is adopted taking into account the mechanical compaction likely to be adopted in the construction stage. Besides, it is assumed that there be no difference between coefficients of permeability of the existing embankment body and that to be embanked to heighten the existing one concerning the Cross Sections TRE-2 and GLE.

The analysis results are shown in Table 5.2 and Figure 5.8. As shown in those Figures, the seepage line crosses with the embankment slope at higher elevation than the ground surface for all the Cross Sections. Hence, the filter zone and/or berm (or low dike) are required to be built in the country side of the embankment body, if the seepage flowing from the river side in the high river stage is to reach the embankment slope of the country side within the monsoon period.

b) Reaching time of seepage line to embankment slope in country side

The time for the seepage to reach from the river side to embankment slope of the country side on each of the typical Cross Sections was estimated by the following formula developed by Strahl;

$$T = (1/k) \times [\ (b + n'H) \times \{H^2 + (b + n'H)^2\}^{0.5}\ ] / H$$

where,
- T : Reaching time of seepage line to embankment slope on country side (hours)
- k : Coefficient of permeability (m/h)
- b, n' : Factors depending on shape of embankment cross section
- H : Hydraulic head (m)

The time was analyzed for the following cases;

Case-1 : The inland water level is equal to the ground surface
Case-2 : The inland water level is at the design inland water level (DIWL)

From the stage hydrographs of the Teesta and Ghagot in 1987, the high stages of those rivers are assumed to last for three months. Reaching time of seepage line to embankment slope on the country side was calculated to be 123 to 389 days on TRE and 57 to 79 days on GLE. The analysis results are shown in Table 5.3. As seen in the Table, the seepage line on the Section GLE reaches on the embankment slope in the country side within the monsoon period, whereas it doesn't appear within the period with respect to the Cross Sections TRE-1 and TRE-2. Therefore, the filter zone is required to be provided in the flood embankments along the Ghagot.

c) Piping resistance

The piping resistance, that is to say, a possibility of occurrence of piping failure, is examined using the method based on the critical hydraulic gradient. The critical hydraulic gradient is estimated for each of the typical Cross Sections using the following K. Terzaghi/L.F. Harza's formula, in which the critical gradient against piping is expressed by the relation between void ratio and specific gravity of foundation soil layer;

$$i_c = (G_s - 1) / (1 + e)$$

where,
- i_c : Critical hydraulic gradient
- G_s : Specific gravity of soil particles
- e : Void ratio of soils

The analysis is made on the worse condition that there is no inundation in the country side, since the hydraulic gradient decreases with the inland water level. The analysis results are shown in Table 5.4, which shows that the hydraulic gradients along the typical Cross Sections are much smaller than the critical hydraulic gradient concerning all the examined cases. Therefore, those flood embankments are judged to have the sufficient piping resistance.

(3) Settlement analysis

The settlement analysis was carried out to estimate total settlement, which means a sum of settlements in the embankment body itself and existing soil layers below that. Besides, the settling velocity of the flood embankments was made.

a) Total settlement

With regards to the flood embankments to be provided in the GIP area, it is foreseen that the settlement

will occur in three different soil portions, namely; settlement in the new embankment body itself, the upper silty soil layer and lower sandy soil layer. However, it is considered that the major settlement will take place in the new embankment and the silty soil layer only, since an extent of the settlement in the lower sandy soil layer is insignificant due to its geotechnical characteristics detected through the geotechnical investigations. The settlements in each of the embankment zone and the silty soil layer is estimated applying the e-log p curve shown in Figure 5.5 to the following formula;

$$S = (e_o - e) / (1 + e_o) \times H$$

where,
- S : Settlement (cm)
- e_o : Initial void ratio
- e : Void ratio after construction of the embankment
- H : Thickness of compressible stratum (cm)

Then, the total settlement of the flood embankments is estimated by adding the settlements in the both portions;

$$Sf = Se + Sm$$

where,
- Sf : Total settlement (cm)
- Se : Settlement in the embankment portion (cm)
- Sm : Settlement in the silty soil layer (cm)

With respect to the existing flood embankment to be heightened and/or resectioned, the analysis is made on the condition that the existing embankment body and its foundation soil layer have been settled before commencement of the construction. The analysis results are shown in Table 5.5. As seen in the Table, the settlements in the embankment body are estimated to be relatively as small as 3.1 to 3.6 cm in the Sections TRE-1 and -2, and 2.4 cm in the Section GLE. On the other hand, the settlements in the foundations soil layers much differ place by place, mainly depending upon thickness of the silty soil layer and embankment height, which range from 4.5 to 29.4 cm in the Sections TRE-1 and -2 and 3.4 cm in the Section GLE.

b) Settling velocity

The settling velocity is analyzed only for the silty soil layer, since the settlement in embankment be mostly completed during embankment works considering the drainage conditions. The Terzaghi's one dimensional consolidation theory expressed by the following formula is used to estimate the settling velocity of the silty soil layer,

$$T = c_v / H^2 \times t$$

where,
- T : Time factor
- c_v : Coefficient of consolidation (cm²/day)
- H : Minimum thickness of drainage (cm)
- t : Time (day)

In this analysis, it is considered that one side drainage takes place in the sandy soil layer only. Besides, the gradual consolidation of the foundation soil layer caused with increase of the load during the embankment construction is also taken into account assuming the construction period of 6 months (180

days).

The analysis results are shown in Table 5.5 and Figure 5.9. The estimated residual settlement of the foundation is estimated to be 23.9 cm for the Section TRE-2, whereas those for the Sections TRE-1 and GLE are less than 2.1 cm. In order to cope with the settlement of the flood embankment which is expected to take place during and after construction thereof, hence, every type of the flood embankment needs to be so constructed as to have an extra height above the design crest level along the whole section. It is recommended that the minimum extra embankment heights be taken at 30 cm for the Section TRE-2 and at 10 cm for the Sections TRE-1 and GLE, taking into consideration the settlement which might be caused by other inconceivable factors during and after the construction.

(4) Slope stability analysis

The stability analysis of the flood embankments is made applying the following Fellenius method (or called the Sweden method);

$$F_s = \Sigma\ (c' \cdot l + (N - U - N_e) \tan f') / \Sigma(T + T_e)$$

where, F_s : Safety factor
N : Normal force acting on slip circle of each slice
T : Tangential force acting on slip circle of each slice
U : Pore water pressure acting on slip circle of each slice
N_e : Normal force of extreme loading condition acting on slip circle of each slice
T_e : Tangential force of extreme loading condition acting on slip circle of each slice
c' : Cohesion of materials
l : Arc length of slip circle of each slice
f' : Internal friction angle of flood embankment materials

The minimum required safety factor for the stability of the flood embankments is adopted to be 1.25 on the normal loading condition and 1.10 on the extreme loading condition. The stability analysis is carried out for the following cases which are depicted in Figure 5.10;

Case No.		Condition
Case-1	:	Immediately after construction of embankment when ground surface in the both river and country sides becomes dry.
Case-2	:	Water levels in the river and country sides are at the design high water level of the river (DWL) and design inland water level (DIWL), respectively.
Case-3	:	Water level in the river side is at DHWL, while water level in the country side is at the ground elevation.
Case-4	:	Rapid drawdown from DWL to the ground level takes place in the river side, when water level in the river side is at DIWL.
Case-5	:	Rapid drawdown from DWL to the ground elevation takes place, while water level in the country side is at the ground elevation.
Case-6	:	Both water levels in the river and country sides are at DIHL.
Case-7	:	Water level in the river side is at DIHL, while water level in the country side is at the ground elevation.

Concerning the seismic force taken into account under the extreme conditions, 50 % of the design seismic coefficient (0.55) is applied to the Cases-1 to -5, since it is considered very rare that earthquake of design magnitude occurs under those conditions. While, the design seismic coefficient (0.11) is adopted in the Cases-6 and -7. For the other Cases, no seismic force is considered. It is assumed that seepage line reaches embankment slope on the country side employing the worse condition.

Taking into account a hauling distance of the embankment material in the construction stage, the borrow areas are to be acquired in the river side adjacent to the location of the embankment site. As such being, the stability of the flood embankment will be adversely effected in case the borrow areas be located excessively close to toe of the embankment slope. Hence, to determine the adequate distance between the embankment toe and edge of the borrow area in view of the embankment stability, the stability analysis in the above Case-4 being the most critical among the seven ones is made for the different distances of 3,6,10 and 15 m. The analysis results are summarized in Table 5.6, which indicates that the embankment stability is ensured in case of the distance being taken at 10 m. Therefore, the minimum distance between embankment toe and edge of borrow area is adopted to be 10 m.

In succession, with respect to each of the typical Cross Sections the stability analysis are carried out for the aforesaid seven cases. The results are shown in Table 5.7 and Figures 5.11 to 5.13. As seen in the Table and Figures, all the typical cross sections of the flood embankments are verified to satisfy the required safety factors.

5.3.2 Stress Analysis of Foundation Soil at Planned Regulator Site

The bearing capacity of regulator foundations are examined for the data collected at the boring holes B3, B4, B6',B12 and B13, where provision of regulator was planned in the FAP 2 Interim Report. Although the regulators planned for the GIP in the interim regional study stage have been revised in the final layout plan thereof with respect to their locations and dimensions, it is considered very useful to grasp the values of the allowable bearing capacity for the subsurface soil in the GIP area.

(1) Estimate of Bearing Capacity

a) Basic design values and conditions adopted

The physical properties including specific gravity, natural moisture content, dry, wet and submerged densities, which are required for the estimate of the allowable bearing capacities, are derived based on those obtained through the laboratory and in-situ tests in the geotechnical investigations as shown in Table 5.7. The major dimensions of the regulators are summarized in Table 5.8.

The silty soil layer contains cohesive soil, whereas the sandy soil layer is cohesionless soil layer. It is assumed that the soils in the both said layers are sufficiently saturated. Therefore, the observed N values are corrected to satisfy the conditions of the saturated silty sand soils using the empirical formula by Terzaghi-Peck (1948) in order to estimate those at depth corresponding to two times of a short side of rectangle foundations. The shear strength parameters (c : cohesion, and f : internal friction angle) thereof are calculated using the following formula expressed by the corrected N values;

- For cohesive soil $c = N / 16$ (kgf/cm^2) $f = 0$ (degree)
- For cohesionless soil $c = 0$ (kgf/cm^2) $f = (12\ N)^{0.5} + 15$ (degrees)

b) Calculation formula

The allowable bearing capacity is expressed by the following formula;

$$Q_a = q_d / sf$$

where, Q_a : Allowable bearing capacity of soil layer beneath the structure
q_d : Ultimate bearing capacity of soil layer beneath the structure
sf : Safety factor (sf=3 under the normal condition, while sf=1.5 under the seismic condition.)

To estimate the ultimate bearing capacity, the following Terzaghi's formula is applied in case the general shear failure is likely to occur in the foundation soil layer;

$$q_d = a\, c\, N_c + b\, t_1\, B\, N_t + t_2\, D_f\, N_q$$

where, q_d : Ultimate bearing capacity (tf/m^2)
c : Cohesion below base of foundation (tf/m^2)
t_1 : Unit weight of soil below base of foundation (tf/m^3)(submerged unit weight is taken for soil layer below groundwater table)
t_2 : Unit weight of soil above base of foundation (tf/m^3)(submerged unit weight is taken for soil layer below groundwater table)
a, b : Shape factors of foundation
N_c, N_t, N_q : Bearing capacity factors shown in Figure 5.5
D_f : Depth of foundation (m)
B : Width of foundation (m)

While, the following formula is adopted in case thickness of the cohesive soil layer (H) is less than ($B / 2^{0.5}$), since there is a possibility that the squeeze failure in the foundation soil layer which differs from the general shear failure defined by Terzaghi's theory takes place;

$$q_d = 4\, c_u + c_u\, B / 2\, H$$

where, c_u : Undrained shear strength (tf/m^2)
H : Thickness of cohesive soil layer (m)

c) Estimated allowable bearing capacities

The ultimate bearing capacities at the respective locations, estimated by means of the aforesaid formula, are shown in Tables 5.10 and 5.11 together with the basic design values adopted, and their allowable bearing capacities are summarized as follows;