THE STUDY ON ADDIS ABABA
FLOOD CONTROL PROJECT

CHAPTER 5

URBAN DRAINAGE PLAN

THE STUDY

ON

ADDIS ABABA FLOOD CONTROL PROJECT

IN

THE FEDERAL DEMOCRATIC REPUBLIC OF ETHIOPIA

CHAPTER 5 URBAN DRAINAGE PLAN

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5. URBAN DRAINAGE PLAN

5.1 General

The objective area of urban drainage improvement is the run-off basin which drains to the Bantyiketu river in the reaches from the Finfine bridge up to the confluence of the Kechene and Kurtume rivers. The area is shown in Figure 5.1.1

5.2 Basic Concept

Basic concept on drainage plan to be prepared here is that the drainage facilities are to be additionally constructed to the existing drainage facilities to decrease the inundation conditions in the target area due to the insufficient conditions of the existing drainage facilities.

Rehabilitation of the existing drainage facilities may substantially decrease the present inundation conditions in the target area. But due to the lack of the information, data and drawings of the present facilities, planning of the rehabilitation of the existing facilities could not be achieved in sufficient level. Besides, the project scale of the rehabilitation of the existing facilities may exceed the limited financial sources because all the existing facilities are constructed underground and the costs of the inventory works and the demolishing work of present facilities may pile up too much.

5.3 Basic Policies

Basic policies of drainage plan in the target area are proposed for each sub-basin as follows:

5.3.1 Northern Basin

The run-off from the Northern basin should be trapped just at the northern side of the Saba Square before entering into the rather low-lying area to decrease the drainage discharge to the Bantyiketu river from the target area.

In order that, a drainage ditch with grating across the Churchil Avenue should be constructed on the northern side of the Saba Square. Then the trapped road surface water should be leaded to the Kurtume river before flowing into the culvert under the Churchill Avenue. The location of the drainage ditch is shown in Figure 5.1.1.

5.3.2 Eastern Basin

(1) Sub-basin-E1

The run-off from the sub-basin-E1 should be trapped before passing over the Finfine bridge to decrease the drainage discharge to the Bantyikeri river from the target area.

In order that, a drainage ditch with grating across the Menelik II Avenue should be constructed on the northern side of the Finfine bridge. Then the trapped road surface water should be leaded to the Bantyiketu river just at the bridge site. The location of the drainage ditch is shown in Figure 5.1.1.

(2) Sub-basin-E2

The run-off from the southern portion of the sub-basin-E2 should be collected in front of the Finfine National Restaurant on the Yohanis Street and be leaded to the Bantyiketu river along the planned Bantyiketu Regulation Pond.

From the topological conditions of the Yohanis Street and the surrounding area, the run-off in this basin collects in front of the Finfine National Restaurant. Accordingly new drainage ditch should be constructed in front of the said restaurant across the Yohanis street and the road that meets the Yohanis street in front of the Finfine National Restarurant. The location of the drainage ditch is shown in Figure 5.1.1.

(3) Sub-basin-E3

As mentioned in the chapter on the present conditions of the project area, there exists no serious drainage problem in this area. Accordingly no plan of drainage improvement is proposed.

5.3.3 West-southern Basin

(1) Sub-basin-WS1

The run-off from the sub-basin-WS1 should be trapped on the Ras Danitew Street and be drained to the Bantyiketu river before joining the run-off from the sub-basin-WS2 to decrease the drainage discharge to the Bantyiketu river from the low-lying target area.

In order that, a drainage ditch with grating across the Ras Danitew Street about 125 m southern side of the crossroads of the Yohanis Street and the Ras Danitew Street should be constructed to drain the run-off water to the Bantyiketu river on the upstream side of the Cottage Restaurant about 60 m downstream of the confluence of the Kechene and Kurtume rivers. The location of the drainage ditch is shown in Figure 5.1.1.

(2) Sub-basin-WS2

The run-off from the sub-basin-WS2 should be discharged to the Bantyiketu river by additional drainage ditch around the Addis Ababa Stadium and along the Ras Danitew Street. The location of the drainage ditch is shown in Figure 5.1.1.

(3) Sub-basin-WS3

The run-off from the sub-basin-WS3 should be discharged to the Bantyiketu river by additional two drainage ditches; one across the street that meets the Ras Mekonin Avenue just on the western side of the Abiot Squae, across and along the Ras Danitew Street, and to the Bantyiketu river, the other acorss and along the Ras Mekonin Avenue in front of the Abiot Square. The location of the drainage ditches ae shown in Figure 5.1.1.

5.4 Design Discharge

5.4.1 Basic Condition and Assumption

As mentioned in the Interim Report, the design discharge is to be determined on the condition that the rainfall intensity is 30 mm/hour that corresponds to the return period between 1 and 2 years.

Since the drainage improvement is to be achieved with additional drainage facilities, design run-off is to be drained by the existing facilities and the planned additional facilities. Some portion of the run-off is to be drained by the existing facilites and the remaining portion is to be drained by the additional facilities. The run-off discharge is calculated by using the rational formula. The design discharge for the planned new facilities is determined based on the following conditions and assumptions:

5.4.2 Northern Basin

1) catchment area : 0.25 km^2
2) run-off coefficient : 0.65
3) design discharge : 1.4 m^3/s
4) design discharge for new facilities : 0.7 m^3/s

5.4.3 Eastern Basin

(1) Sub-basin-E1

1) catchment area : 0.23 km^2
2) run-off coefficient : 0.70
3) design discharge : 1.4 m^3/s
4) design discharge for new facilities : 0.7 m^3/s

(2) Sub-basin-E2

1) catchment area of the southern portion : 0.25 km^2
2) run-off coefficient : 0.65
3) design discharge : 1.4 m^3/s
4) design discharge for new facilities : 1.4 m^3/s

Since the existing drainage facilities are presently passing under the grassland that is planned for the flood control regulating pond, all the drainage facilities should be unified into one drainage system from the Finfine National Restaurant site to the Bantyiketu river. Accordingly the design discharge should be the one for this sub-basin.

5.4.4 West-southern Basin

(1) Sub-basin-WS1

1) catchment area : 0.53 km^2
2) run-off coefficient : 0.65
3) design discharge : 2.9 m^3/s
4) design discharge for new facilities : 1.5 m^3/s

(2) Sub-basin-WS2

1) catchment area : 0.54 km^2
2) run-off coefficient : 0.65
3) design discharge : 3.0 m^3/s
4) design discharge for new facilities : 1.5 m^3/s

(3) Sub-basin-WS3

1) catchment area of western portion : 0.39 km^2
2) run-off coefficient : 0.65
3) design discharge : 2.2 m^3/s
4) design discharge for new facilities : 1.1 m^3/s

1) catchment area of eastern portion : 0.17 km^2
2) run-off coefficient : 0.65
3) design discharge : 1.0 m^3/s
4) design discharge for new facilities : 0.5 m^3/s

5.5 Drainage Facilities

In consideration of easy maintenance of the facilities, drainage ditch is proposed to be basically an open channel. And in consideration of the traffic on the ditch, the open channel should be provided with grating. The detail of the drainage facilities of each basin is discussed hereunder.

5.5.1 Northern Basin

A drainage ditch with grating across the Churchil Avenue should be constructed and the leading channel to the Bantyiketu should be cosntructed along the existing road wall to the Bantyiketu river.
The necessary dimensions are hydraulically as follows:

1) width : 0.8 m
2) depth : 0.6 m
3) longitudinal slope : 1/100 (across the avenue), 1/20 (leading channel to the river)

In consideration of maintenance works, the width of 0.8 m is proposed.

5.5.2 Eastern Basin

(1) Sub-basin E1

A drainage ditch with grating across the Menelik II Avenue just upstream side of the abutment of the Finfine bridge should be constructed and the leading channel to the Bantyiketu river should be constructed. The necessary dimensions are hydraulically as follows:

1) width : 0.8 m
2) depth : 0.6 m
3) longitudinal slope : 1/100

The collected surface water should be drained to the highwater channel of the Bantyiketu river where open drain channel is presently available. The open channel should have the same carrying capacity with that of the drainage ditch across the avenue. But the grating over the ditch will not be needed.

(2) Sub-basin E2

A drainage ditch with grating across the Yohanis Street and across the street which meets the Yohanis street in front of the Finfine Natioanl Restaurant should be constructed and the leading channel to the Bantyiketu river should be constructed along the proposed Bantyiketu regulating pond. The necessary dimensions are hydraulically as follows:

1) width : 0.8 m
2) depth : 1.1 m
3) longitudinal slope : 1/175

5.5.3 West-southern Basin

(1) Sub-basin WS1

A drainage ditch with grating across the Ras Danitew Street should be constructed and the leading channel to the Bantyiketu river should be constructed between the Cottage Restaurant and the Red Cross that is now under construction. The necessary dimensions are hydraulically as follows:

across the avenue,
1) width : 0.8 m
2) depth : 1.1 m
3) longitudinal slope : 1/100

leading ditch to the river
1) width : 0.8 m
2) depth : 1.2 m
3) longitudinal slope : 1/240

(2) Sub-basin WS2

A drainage ditch with grating across the road that meets the Ras Danitew Street just on the northern side of the Addis Ababa Stadium, along the road, across and along the Ras Danitew Street and the leading channel along the road presently leading to the Addis Ababa Tennis Club should be constructed. The necessary dimensions are hydraulically as follows:

across the avenue
1) width : 0.8 m
2) depth : 1.3 m
3) longitudinal slope : 1/210

along the Ras Danitew Street
1) width : 0.8 m
2) depth : 1.3 m
3) longitudinal slope : 1/132

leading ditch to the river
1) width : 0.8 m
2) depth : 1.55 m
3) longitudinal slope : 1/370

(3) Sub-basin WS3

1) Eastern portion of the sub-basin WS3

A drainage ditch with grating across the road that meets the Ras Mekonin Avenue just on the eastern side of the Abiot Square, across the Ras Danitew Street and the leading channel to the Bantyiketu river should be constructed. The necessary dimensions are hydraulically as follows:

1) width : 0.8 m
2) depth : 0.8 m, 1.0 m, 1.2 m
3) longitudinal slope : 1/400, 1/250, 1/400

2) Western portion of the sub-basin WS3

A drainage ditch with grating across and along the Ras Mekonin Avenue just in front of the Abiot Square should be constructed. The drainage ditch should be connected with the street inlet that is presently over the drainage cluvert connected to the Bantyiketu river. The necessary dimensions of the ditch are hydraulically as follows:

across the avenue
1) width : 0.8 m
2) depth : 0.75 m
3) longitudinal slope : 1/600

along the avenue
1) width : 0.8 m
2) depth : 0.50 m
3) longitudinal slope : 1/95

The longitudinal and cross-sectional profiles of all these drainage ditches are shown in Figures 5.5.1 – 5.5.8.

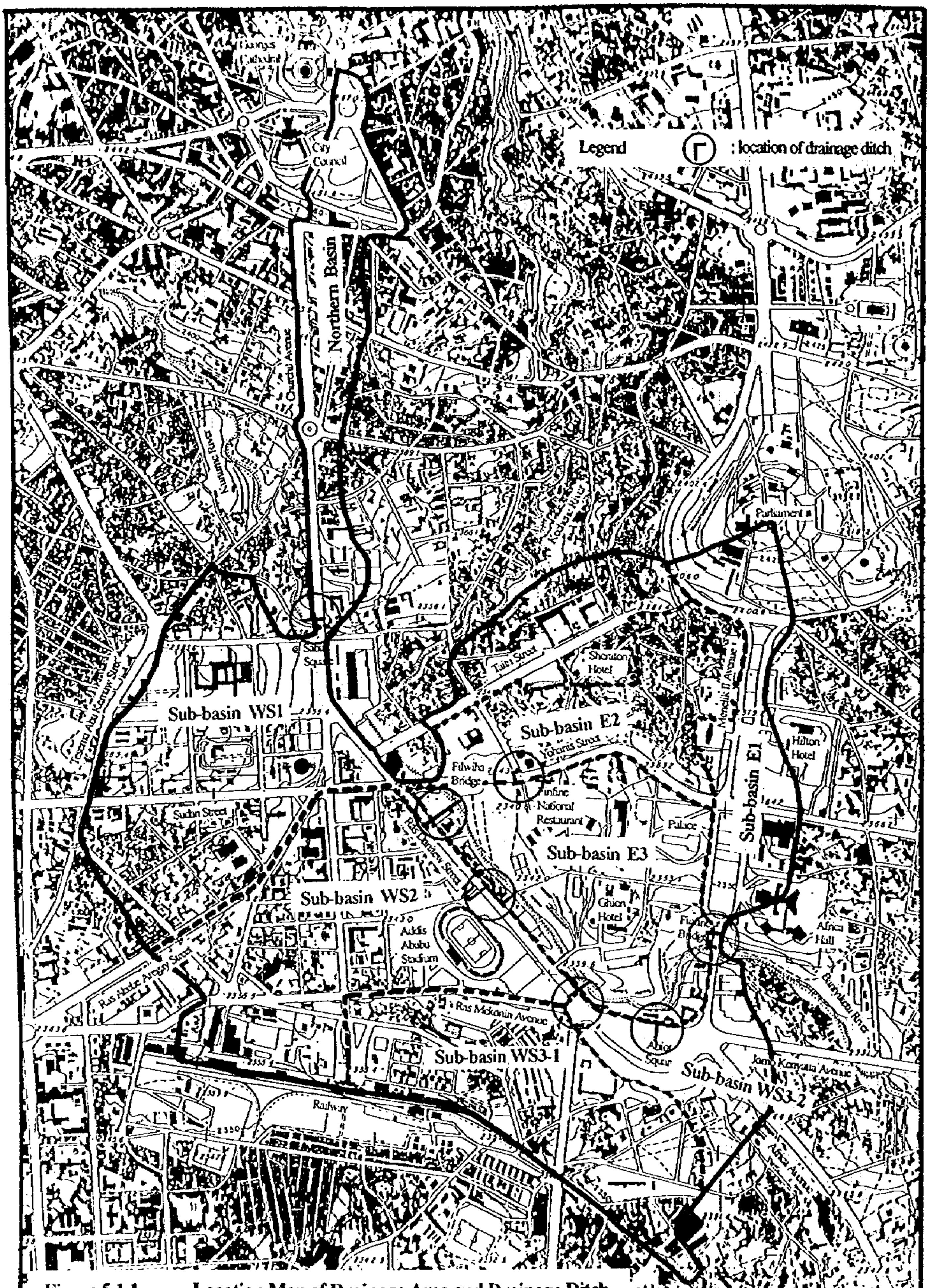

Figure 5.1.1 Location Map of Drainage Area and Drainage Ditch

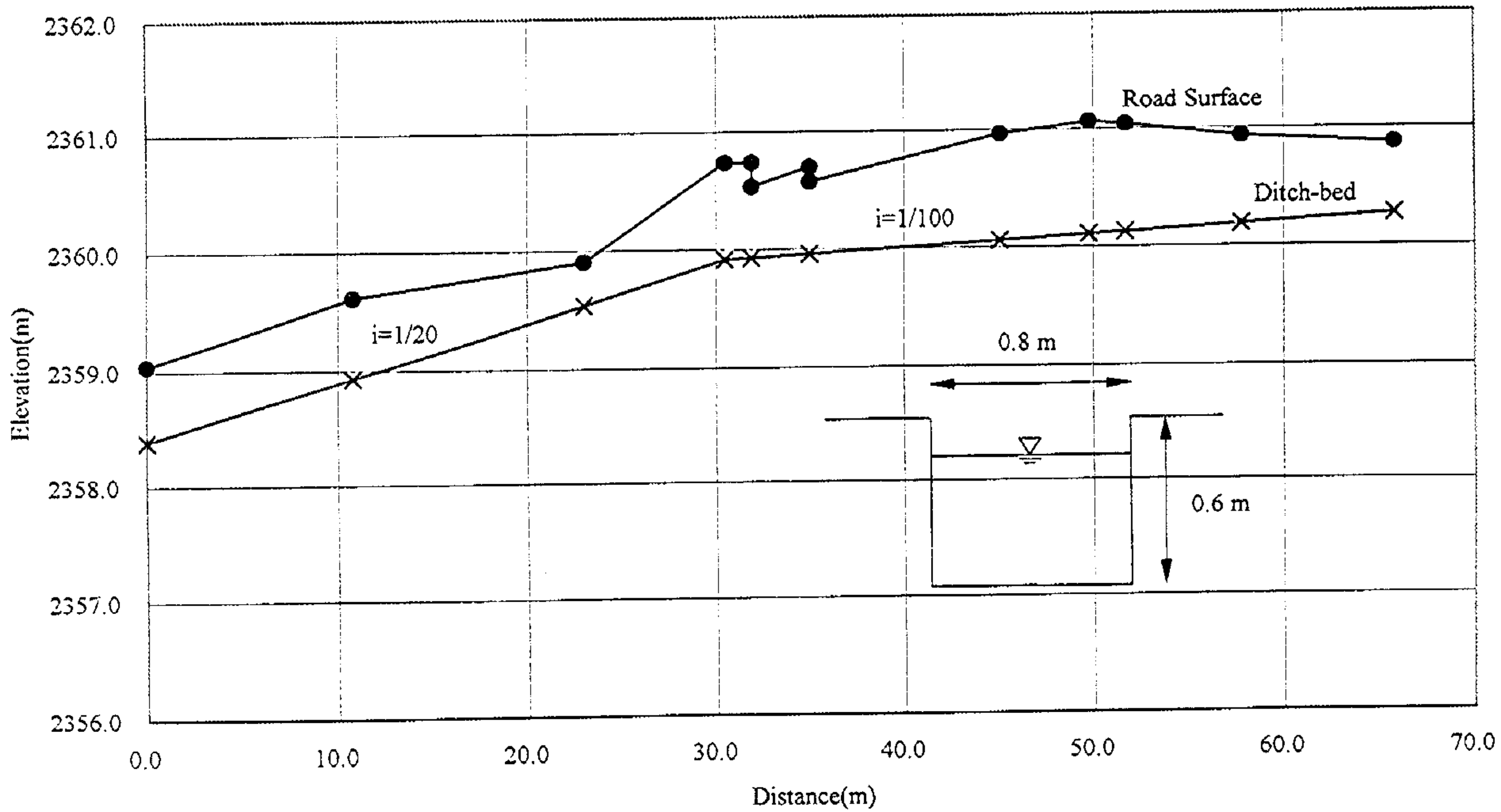

Figure 5.5.1 Longitudinal and Cross-sectional Profiles of Drainage Ditch for Northern Basin

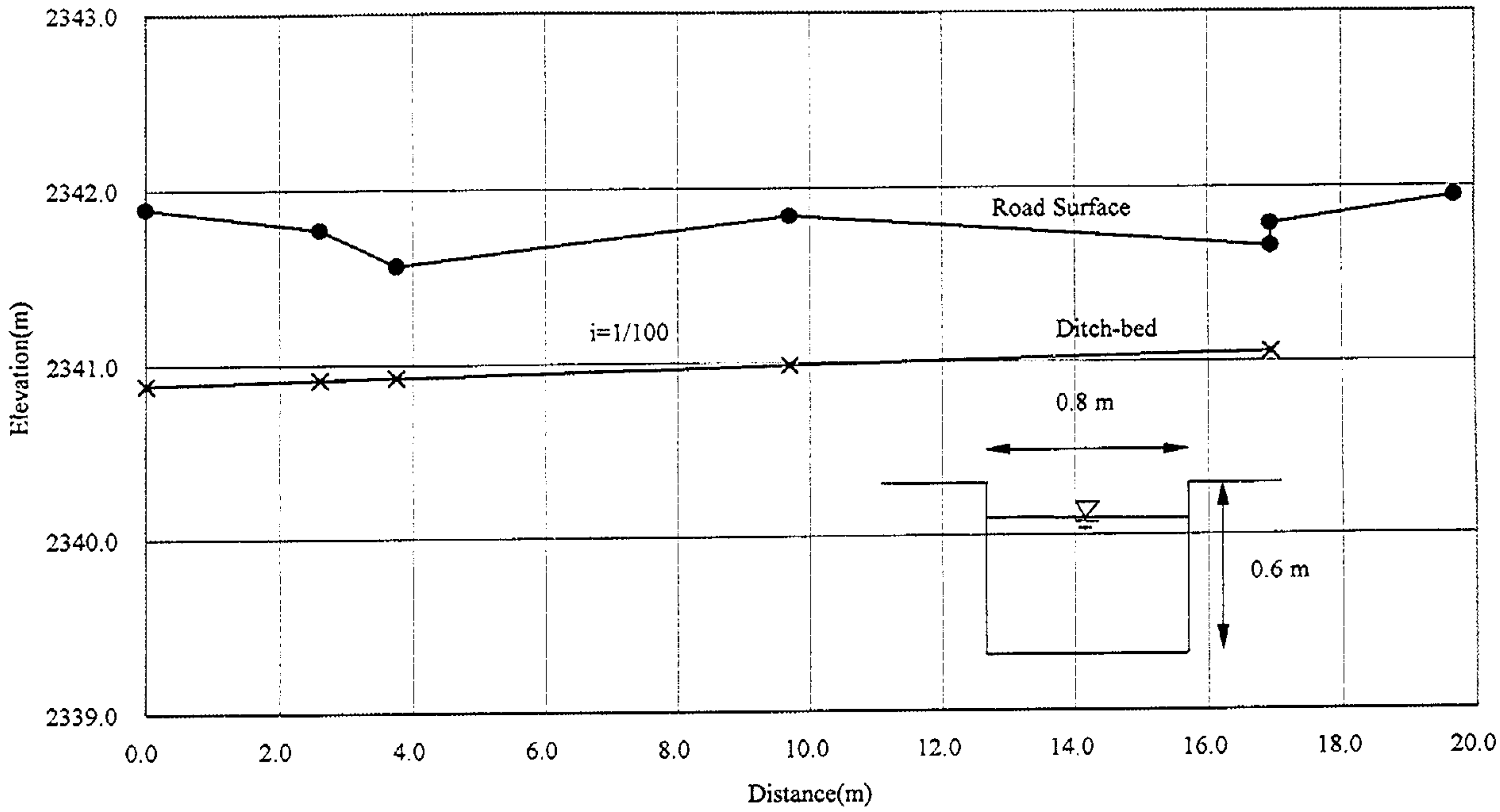

Figure 5.5.2 Longitudinal and Cross-sectional Profiles of Drainage Ditch for E1 Basin

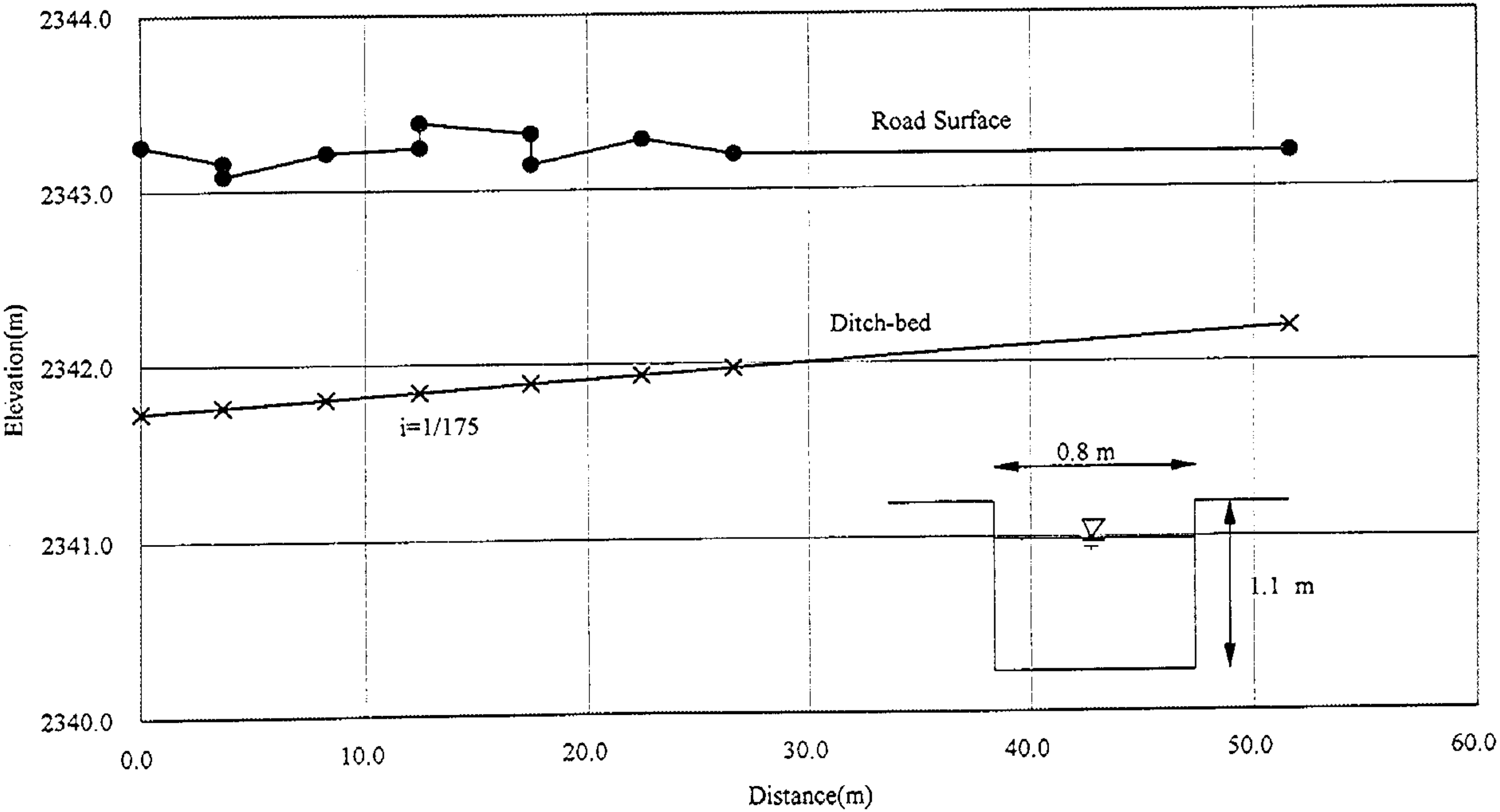

Figure 5.5.3 Longitudinal and Cross-sectional Profiles of Drainage Ditch for E2 Basin

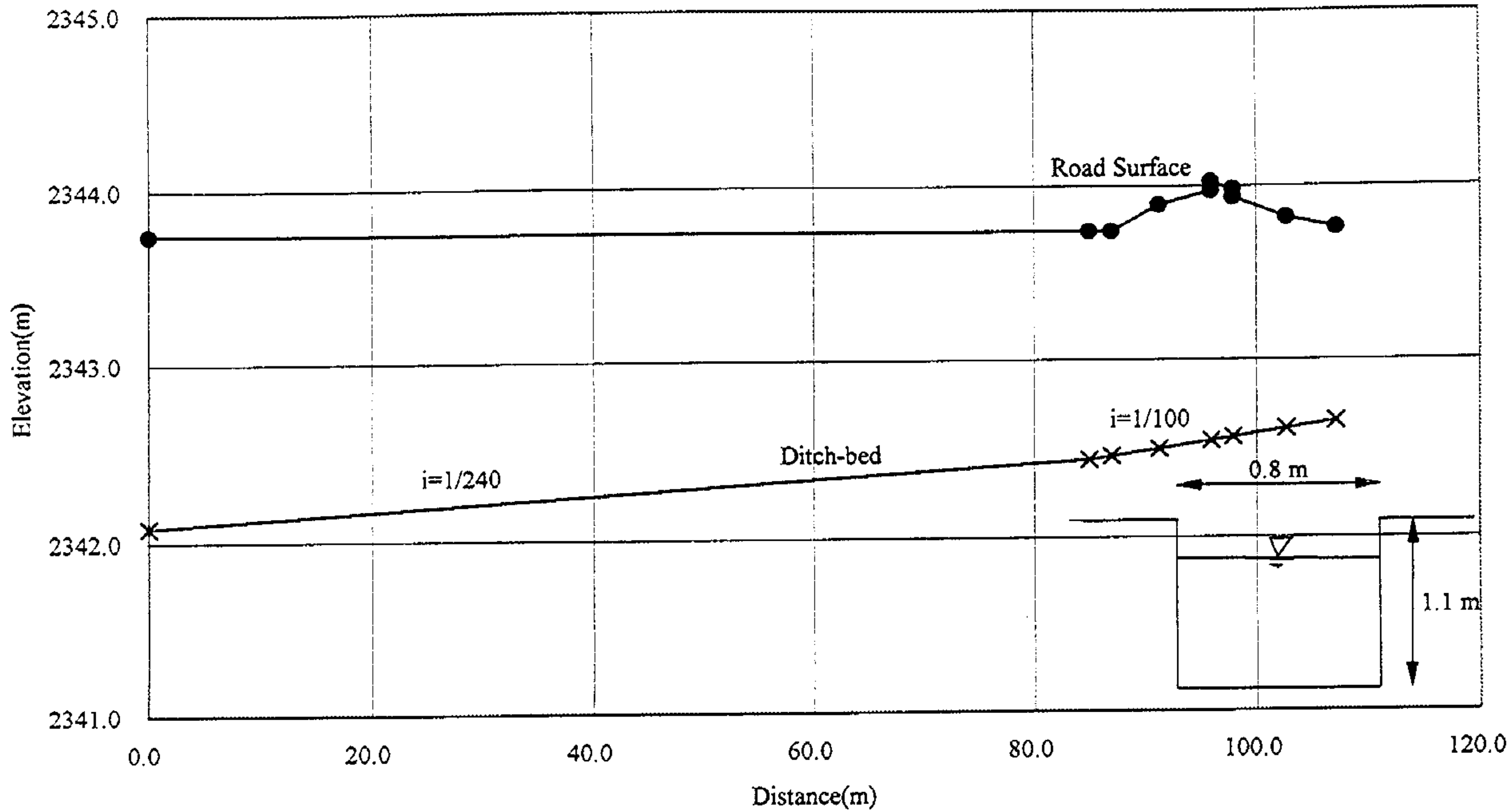

Figure 5.5.4 Longitudinal and Cross-sectional Profiles of Drainage Ditch for WS-1 Basin

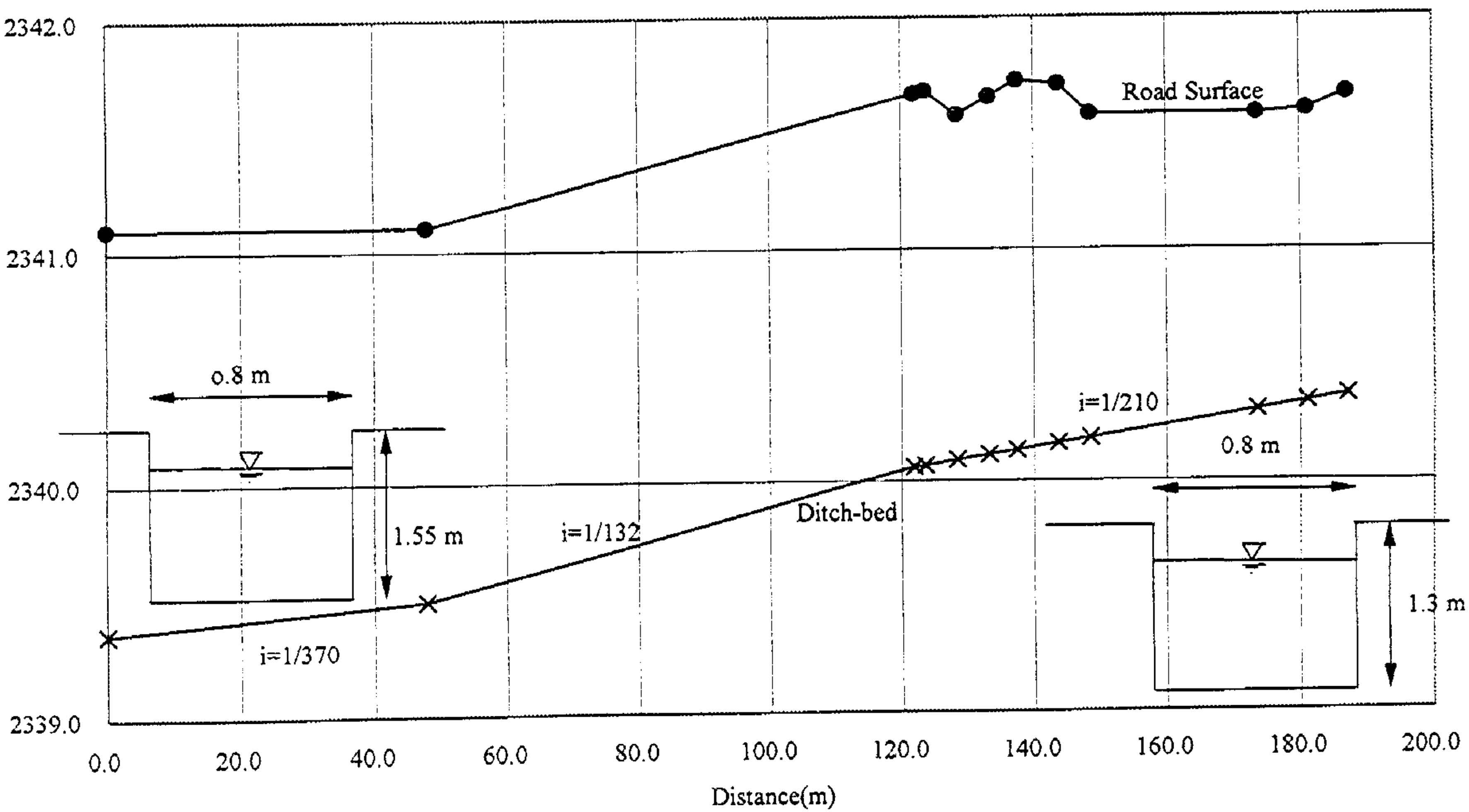

Figure 5.5.5 Longitudinal and Cross-sectional Profiles of Drainage Ditch for WS2 Basin

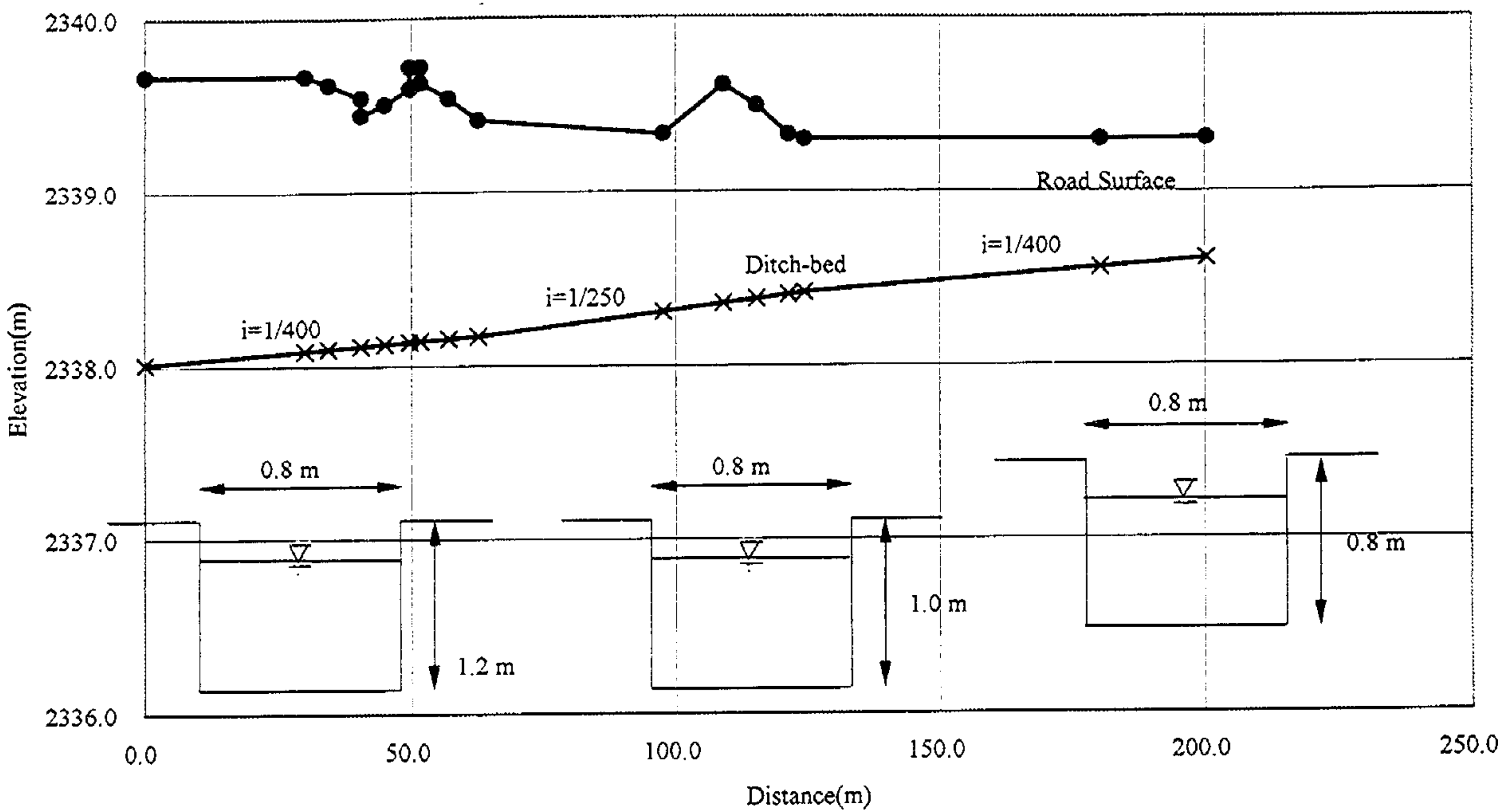

Figure 5.5.6 Longitudinal and Cross-sectional Profiles of Drainage Ditch for WS3-1 Basin

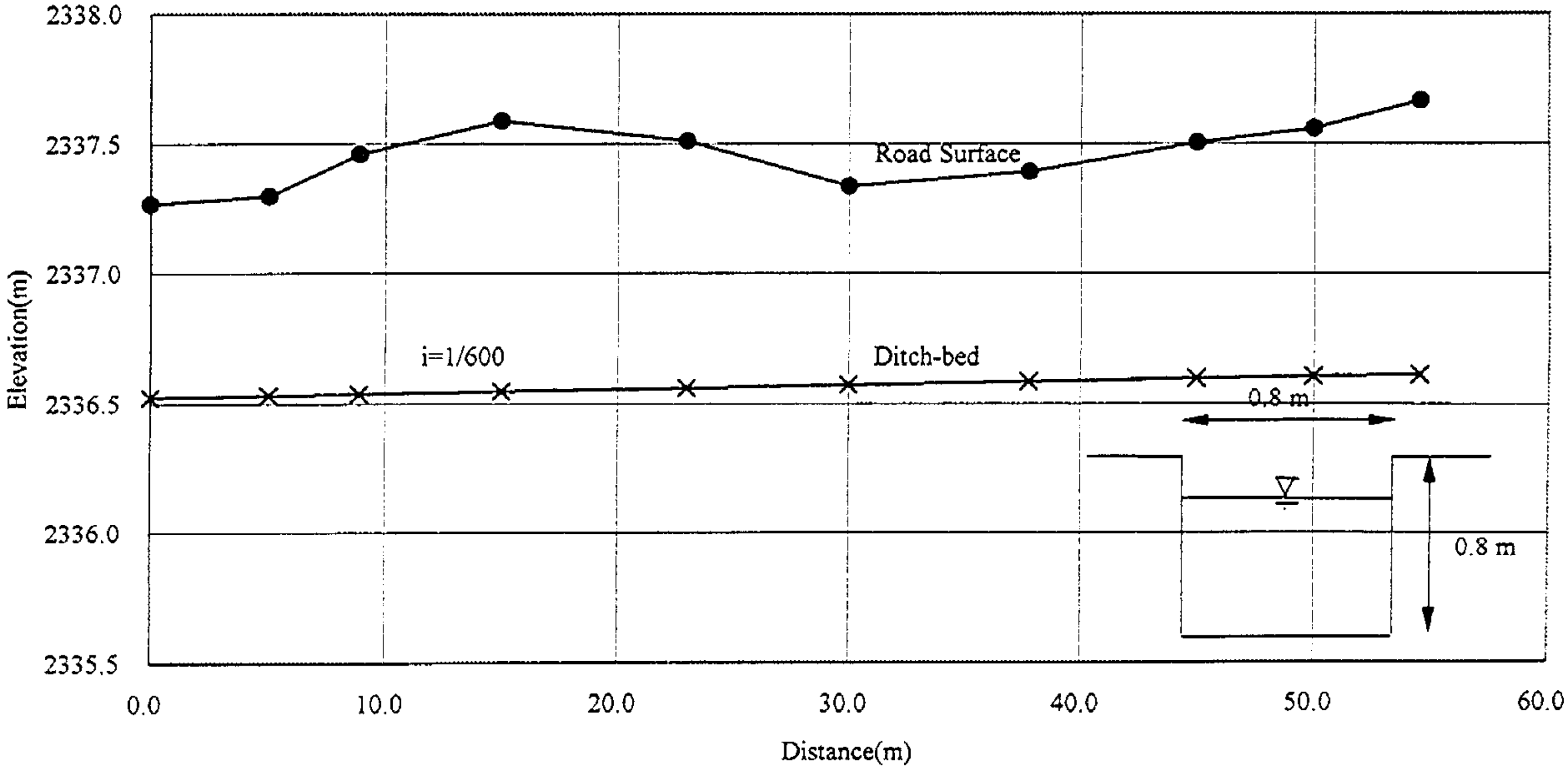

Figure 5.5.7 Longitudinal and Cross-sectional Profiles of Drainage Ditch across the Avenue for WS3-2 Basin

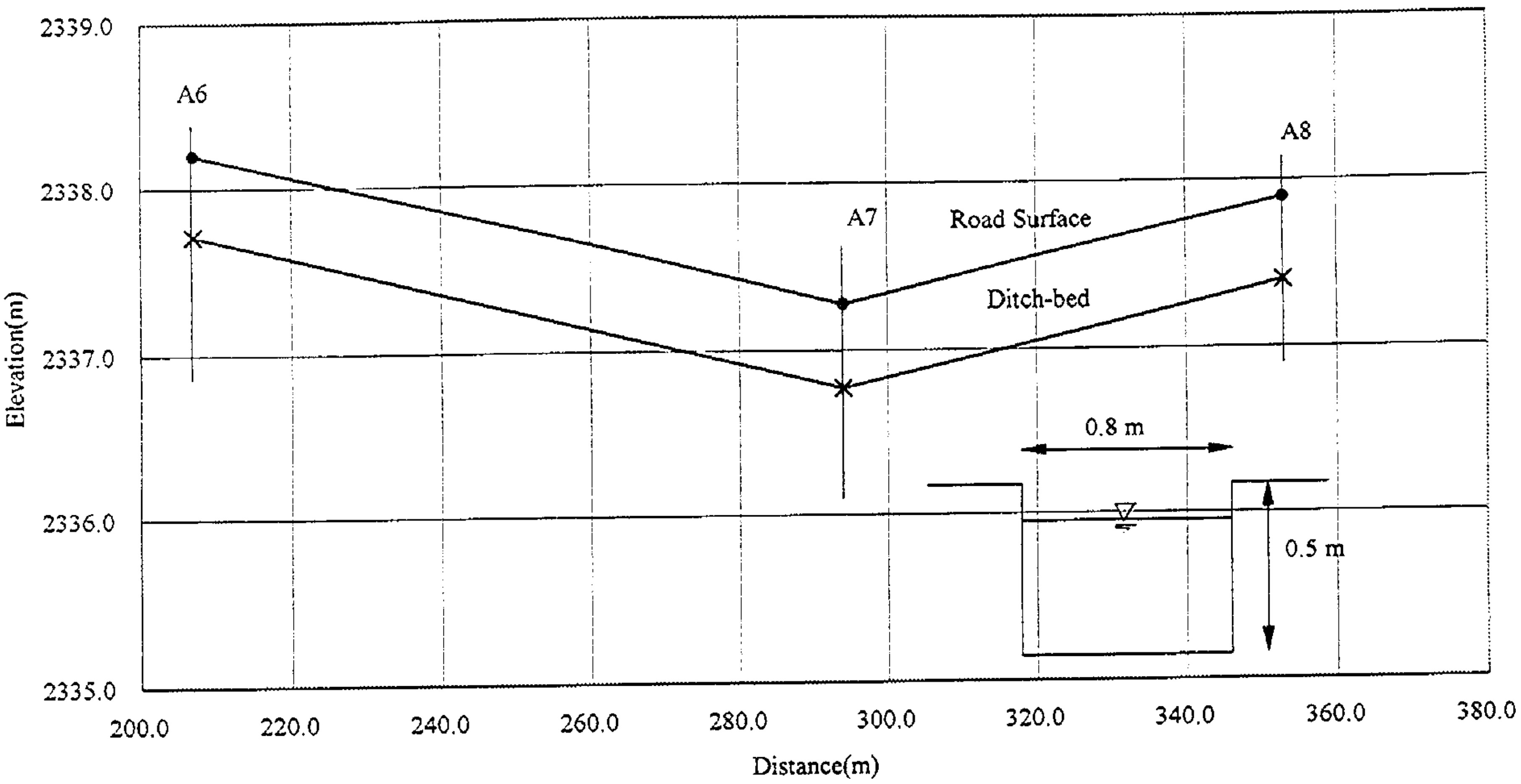

Figure 5.5.8 Longitudinal and Cross-sectional Profiles of Drainage Ditch along Avenue for WS3-2 Basin

CHAPTER 6

DESIGN OF FACILITIES

THE STUDY

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ADDIS ABABA FLOOD CONTROL PROJECT

IN

THE FEDERAL DEMOCRATIC REPUBLIC OF ETHIOPIA

CHAPTER 6 DESIGN OF FACILITIES

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6. DESIGN OF FACILITIES

6.1 Basic Concepts and Methodology

In the feasibility study, the following flood control and urban drainage facilities are designed.

	Location	Remark
Flood Control Facilities		
1. River Improvement		
(1) Earth Dyke	Near the Kebena and Bulbula Vegetable Garden in a downstream stretch along the Bantyiketu river	
(2) Flood Wall	Eight sites at the Bantyiketu river; a site at the Kechene river	
(3) Slope Protection Work	Immediately upstream from the Bole Bridge along the Kebena river; Immediately downstream from the existing irrigation intake weir of the Kebena and Bulbula Vegetable Garden; Immediately upstream from the Bantyiketu bridge (Total 3 sites along the Bantyiketu river)	
(4) Other structures		
1) Irrigation Intake Weir	In the Kebena and Bulbula Vegetable Garden in a downstream stretch of the Bantyiketu river	Due to riverbed excavation, the height of the existing weir is to be lowered. The new weir is to be built upstream from the existing weir site.
2) Aqueduct of a Water Supply Pipe of AAWSA	Near the Asmera Road along the Bantyiketu river	The aqueduct is to be reconstructed to secure freeboard against the design flood discharge.
3) Crossings of Sewerage Pipe beneath Riverbed	Near the National Hotel along the Bantyiketu river	The two crossings are to be reconstructed due to riverbed excavation.
2. Regulating Pond	One at the Kostre river (a tributary of the Kechene river) and the other at Bantyiketu river	
3. Flood Control Weir	In an upstream stretch of the Kechene river	
Urban Drainage Facilities		
1. Drain Ditches	At the Finfine bridge; At the Abiot square; Near the Addis Ababa Stadium; Near the Finfine National Restaurant; Near the Saba Square	

Basic concepts of design are that structures have to be (1) durable, (2) operated unmanned, (3) free from maintenance, and (4) be made of locally available materials.

In design, Ethiopian standards, criteria and design practices are used as much as possible. For facilities or part of facilities including earth dyke, slope protection work, an irrigation intake weir and a flood control weir, of which no relevant Ethiopian standards and criteria are available, Japanese standard and criteria are applied. The standards and criteria applied in design are as follows.

(1) Ethiopian Building Code Standard (1995 edition) published by Ministry of Works and Urban Development

1) EBCS-1, Basis of Design and Actions on Structures
2) EBCS-2, Structural Use of Concrete
3) EBCS-7, Foundations
4) EBCS-8, Design of Structures for Earthquake Resistance

(2) Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan

(3) Free Frame Method of Slope Protection, Guideline of Design and Construction, (Third edition) published in January 25, 1996, by Free Frame Association, Japan

(4) Guideline of Drainage Facilities (June 1987 edition) published by Japanese Road Association, Japan

(5) Standard Design Drawings of Civil Structures No.1 (Drain Ditch, Pipe Culvert, Box Culvert) published by the Ministry of Construction, Government of Japan

6.2 Basic Conditions of Design

6.2.1 Unit Weight

Unit weights are quoted from EBCS-1, Ethiopian Building Code Standard (1995 edition), Basis of Design and Actions on Structures, published by Ministry of Works and Urban Development as shown on Table 6.2.1.

As for embankment materials for earth dyke, their unit weight and other properties such as cohesion and angle of internal friction are specified by neither Ethiopian Building Code Standard (1995 edition) published by Ministry of Works and Urban Development nor Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan.

6.2.2 Seismic Coefficient

The seismic coefficient used for the stability analysis of concrete gravity weirs are calculated on the basis of EBCS-8, Ethiopian Building Code Standard (1995 edition), Design of Structures for Earthquake Resistance, published by Ministry of Works and Urban Development.

According to EBCS-8, the Ethiopian Building Code Standard, the seismic coefficient is given by the following formula.

$$Sd = \alpha \beta \gamma$$

where,

Sd : Seismic coefficient against the acceleration of gravity

α : Ratio of the design bedrock acceleration to the acceleration of gravity given by

$$\alpha = \alpha_0 I$$

where,

α_0 : Bedrock acceleration ratio for the site which depends on the seismic zone as given in Table 6.2.2 and Figure 6.2.1. As Addis Ababa belongs to Zone 2 from Figure 6.2.1, α_0 is equal to 0.05.

I : Importance factor given in Table 6.2.3

Because the concrete gravity weir planned in this study are considered to be quite important, I equal to 1.4 is used.

β : Design response factor given by the following formula.

$$\beta = 1.2 S / T^{2/3}$$

where,

S : Site coefficient for soil characteristics given by Table 6.2.4.

As the planned concrete gravity weir is situated on hard rock, S is assumed as 1.0.

T : Fundamental period of building in second given by the following formula.

$$T = C H^{3/4}$$

where,

C : Coefficient related to the resistance against moment given by :

0.085 for steel moment resisting frames

0.075 for reinforced concrete moment-resisting frames and eccentrically braced frames

0.050 for all other buildings

Because concrete gravity weir is rigid with a large resistance against moment, C equal to 0.085 is used.

γ : Behavior factor which represents the energy dissipation capacity of structures given as a ratio against a completely elastic structure with viscous damping of 5 %.

As the behavior factor of mass concrete structures like concrete gravity weirs is not specified in the Ethiopian Building Code Standard or any other Ethiopian standard. However, the behavior factor of other structures are shown in EBCS-8, Ethiopian Building Code Standard (1995 edition), Design of Structures for Earthquake Resistance as follows:

For concrete buildings : $\gamma \leqq 0.70$

For unreinforced masonry buildings : $\gamma = 0.70$

As a conservative estimate, the behavior factor for concrete gravity weirs is assumed 0.70.

From the above, the seismic coefficient Sd is derived, assuming the weir height is about 20 m, as follows.

$$Sd = \alpha \beta \gamma$$

where,

$$\alpha = \alpha_0 I = 0.05 \times 1.4 = 0.07$$

$$\beta = 1.2\,S / T^{2/3} = 1.2\,S / (C\,H^{3/4})^{2/3}$$
$$= 1.2 \times 1.0/(0.085 \times 20^{3/4})$$
$$= 1.49$$

$$\gamma = 0.70$$

Therefore,

$$Sd = 0.07 \times 1.49 \times 0.70 = 0.07$$

In the stability analysis of concrete gravity weir contemplated by this study, Sd is made equal to 0.10 for the sake of conservativeness.

6.2.3 Sediment

The reservoir sediment volume is used by the stability analysis of concrete gravity weirs, converting sediment volume into sediment depth. In this study, sediment volume is derived from an existing study report entitled "Addis Ababa Water Supply Project – Stage IIIA, Final Design and Tender Documents Preparation, Hydrology of Gerbi Dam, Final Report" undertaken by the Addis Ababa Water and Sewerage Authority, finalized in January 1997.

The proposed concrete gravity weir is designed to have outlet holes, as non-emergency spillway, at the elevation almost as high as the existing riverbed for the purpose of discharging floods equal to or less than design flood (probable 30 year flood) as well as instream flows to downstream areas without manned operation. Therefore, almost no sediment is likely to deposit in the reservoir, being transported downstream by stream flows , as long as the outlet holes are not clogged. In the stability analysis of concrete gravity weirs, reservoir sedimentation is assumed for the worst case in which the outlet holes are completely closed.

Sediment volume applied to the stability analysis of concrete gravity weirs is estimated on the basis of suspended load measured at four sites during the said hydrological study of the Gerbi dam, assuming that (1) the bed load is equal to 10% of the suspended load, (2) the unit weight of deposited sediments in the reservoir is equal to 0.91 tf/m^3, (3) the trap efficiency of reservoirs is 100%, and (4) 100-year sedimentation volume is applied as specified in Design Standard of River and Sabo Structures (September 1997) published by the Ministry of Construction, Government of Japan. The estimated sediment volume of the proposed weir is shown on Table 6.2.5 and summarized as follows.

Sites of which Suspended Load Measurement was Conducted	Gerbi Dam	Chacha	Robi Gomoro	Berga	Average
100-year Sediment Volume of Kechene Weir (m^3) (Catchment Area=5 km^2)	37,000	62,000	63,000	39,000	51,000

6.2.4 Earth Dyke

(1) Crest Elevation

Crest elevation of earth dykes is determined as the elevation above design high water level of rivers with an arbitrary freeboard. The freeboard is stipulated in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan, as follows.

Design Discharge (m^3/s)	Freeboard Above Design High Water Level (m)
$Q < 200$	0.6
$200 \leqq Q < 500$	0.8
$500 \leqq Q < 2,000$	1.0
$2,000 \leqq Q < 5,000$	1.2
$5,000 \leqq Q < 10,000$	1.5
$10,000 \leqq Q$	2.0

(2) Crest Width

Crest width of earth dykes is determined on the basis of Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan, as follows.

Design Discharge (m^3/s)	Crest Width (m)
$Q < 500$	3
$500 \leqq Q < 2,000$	4
$2,000 \leqq Q < 5,000$	5
$5,000 \leqq Q < 10,000$	6
$10,000 \leqq Q$	7

(3) Slope

Minimum slope of earth dykes is to be 2.0 (Horizontal) against 1.0 (Vertical) as stipulated in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan.

(4) Embankment Material

Embankment material of earth dykes is to satisfy the provisions of EBCS-7, Ethiopian Building Code Standard (1995 edition), Foundations, published by Ministry of Works and Urban Development as follows.

In Section 9.4 of Chapter 9 Embankments and Slopes:

> "(3) In general, for embankment of height up to about 4 m, most soils can be used provided that due care is taken with the foundation, compaction, and side slopes."

In Section 5.2 of Chapter 5 Excavation, Fill and Dewatering and Ground Improvement:

> "(4) Suitable fill materials include compactable and well graded natural granular materials. Some cohesive materials may be suitable but require particular care."
>
> "(5) The following aspects shall be considered when selecting a fill material:
>
> (a) gradation, (b) resistance to crushing, (c) compactibility, (d) plasticity, (e) resistance to weathering, (f) organic content, (g) chemical aggressivity, (h) pollution effects, (i) solubility, (j) susceptibility to volume changes (swelling clays and collapsible materials)", and
>
> "(10) The fill material shall not contain organic and foreign matter"

6.2.5 Revetment of Earth Dyke

(1) Crest Elevation of Revetment

Crest elevation of revetment to protect earth dykes is to be as high as the crest of earth dykes as stipulated in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan.

(2) Crest Elevation of Foundation of Revetment

Crest elevation of foundation of revetment is to be either 0.5m to 1.5m below design riverbed or the surface of base rock of riverbed, whichever less deeper, as stipulated in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan.

(3) Structure

The earth dyke is proposed to be built in the brim of a natural flood retarding basin situated immediately upstream from the existing irrigation intake weir of theKebena and Bulbula Vegetable Garden and flow velocity during floods is expected relatively small. For revetment, gabion mattress is to be used, which is durable, flexible and made of stones and wire meshes easily available at localities. Wet masonry is considered unsuitable because it is neither flexible nor responsive to displacement or settlement of earth dykes, resulting in impending failure of revetment and earth dykes.

Protective measures such as rock and gabion mattress are to be placed at the foot of revetment to prevent the destruction of revetment by scouring.

6.2.6 Flood Wall

(1) Crest Elevation of Flood Wall

Crest elevation of flood walls is determined in the same manners with earth dykes as described in the foregoing section "(4) Earth Dyke".

(2) Slope

Slopes of flood walls are determined as follows.

1) The slope of the same side with river : Vertical which is the same as that applied by Addis Ababa Flood Control and Prevention Office (AFCPO)
2) The slope of the opposite side of river : Determined from the stability analysis

(3) Structure

Flood walls are to be made of wet masonry according to the current design practice of Addis Ababa Flood Control and Prevention Office (AFCPO), in which requirements of materials and structures of flood walls are shown as below.

1) Concrete class : C-20
2) Cement content : 320 kgf/m^3

3) Smallest diameter of stone : 30cm

4) Interval of Expansion Joint : Every 15m

5) Weep hole :

- Vertical interval of every 1.5m and horizontal interval of every 2.5m
- Place filter materials around the end of weep holes in the opposite side of river.

6) Crest :

- Crest width equal to 60cm
- Place concrete capping of 10cm in thickness at the crest

6.3 Flood Control Facilities

6.3.1 River Channel Improvement

The locations of facilities related to river channel improvement such as earth dyke, flood wall, slope protection work, an irrigation intake weir and crossing of sewerage pipe beneath riverbed are shown on Figure 6.3.1.

These facilities are described as follows.

(1) Earth Dyke

The proposed earth dyke is to be built at the right bank of the Bantyiketu river, near the Kebena and Bulbula Vegetable Garden. Its major features are as shown below.

Major Features		Remarks
Crest Elevation	0.6m of freeboard above design high water level	In compliance with Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan
Crest Width	3 m	(do)
Slope	1 (Vertical) to 2.0 (Horizontal)	(do)
Height	About 1m	
Length	About 100m	
Material of Dyke	Material available near the proposed construction site	In compliance with EBCS-7, Foundations, Ethiopian Building Code Standard (1995 edition) published by Ministry of Works and Urban Development
Material of Revetment	Gabion Mattress	

The earth dyke is proposed in the brim of a natural flood retarding basin situated immediately upstream from the existing irrigation intake weir of the Kebena and Bulbula Vegetable Garden and flow velocity during floods is expected small. Hence, gabion mattresses are to be used for revetment of earth dyke

The typical cross-section of earth dyke is shown on Figure 6.3.2.

(2) Flood Wall

The flood walls, designed based on the current design practice of Addis Ababa Flood Control and Prevention Office (AFCPO), are to be made of wet masonry and be built at eight sites of the Bantyiketu river and at a site of the Kechene river. Their major features are as follows.

Major Features		Remarks
Crest Elevation	0.6m of freeboard above design high water level	In compliance with Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan
Height	1m to 4 m at the Bantyiketu river	
	4.5 m at the Kechene river	
Length	About 1,100 m at the Bantyiketu river	
	About 120 m at the Kechene river	
Slope	Vertical at the same side with river	In compliance with AFCPO's design practice
	1 (Vertical) to 0.4 (Horizontal)	Determined from the stability analysis in compliance with AFCPO's design practice
Material	Wet masonry	In compliance with AFCPO's design practice

The typical cross-section of flood wall is shown on Figure 6.3.3.

(3) Slope Protection Work

Slope protection work is proposed to protect from torrent during floods the mild slopes of silty or clay soils at three sites in residential areas, namely, immediately upstream from the Bole Bridge along the Kebena river, immediately downstream from the existing irrigation intake weir of the Kebena and Bulbula Vegetable Garden and immediately upstream from the Bantyiketu bridge.

Flood flows at the three sites are expected to become too turbulent and swift to apply gabion mattress. Revetment of wet masonry, though it is rigid, is less flexible so that there is a great chance of creating voids behind revetment, liable to the collapse eventually. For this reason, the free frame method is proposed for slope protection. Extensively used for the protection of slopes, including those along rivers, reservoirs, roads, and in residential areas in Japan, the free frame method consists of grids of cement mortar with square cross-sections, armored by reinforcing bars, and concrete of about 15 cm in thickness filled between grids. To prevent loss of soils behind the slope protection work by suction, mats made of non-woven polyester is placed underneath. Weep holes of galvanized steel are provided.

Major features of slope protection work are as follows.

Major Features		Remarks
Method of Slope Protection	Free Frame Method	
Frame		
Cross-sectional Dimensions	0.3 m x 0.3m (Square)	In compliance with Free Frame Method of Slope Protection, Guideline of Design and Construction, (Third edition) published in January 25, 1996, by Free Frame Association, Japan
Length between a center of frames	2.3 m	(do)
Reinforcing bars in a cross section	D16 x 4 pieces	(do)
Reinforcing bars for stirrup	D13	(do)
Anchor Pin (Reinforcing bar)	D16, L=1m	Placed at every node and a middle of nodes along frames
Filling between frames	Concrete of 15 cm in thickness	
Filter mat	Made of non-woven polyester	
Weep hole	1 piece every $2m^2$; Made of galvanized steel with inside diameter of 5cm	In compliance with Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan

The typical plan and cross-section of slope protection work is shown on Figure 6.3.4.

(4) Other Structures

1) Irrigation Intake Weir

There exists an irrigation intake weir made of wet masonry, about 5 m in height and about 20 m in crest length, in a downstream stretch of the Bantyiketu river to divert water to the Kebena and Bulbula Vegetable Garden.

The crest elevation of overflow section of the existing weir is measured at 2,312.86 m based on the topographic survey and has to be lowered to design riverbed elevation of 2,311 m determined by this study. For this reason, the crest of the existing irrigation intake weir is to be demolished and a new weir made of wet masonry is to be built about immediately upstream from the existing weir site to insure water supply to the Kebena and Bulbula Vegetable Garden.

Major features of a new irrigation intake weir and a new irrigation canal are as follows.

Irrigation Intake Weir	
Height	2 m
Crest Length	30 m
Construction Material	Wet Masonry
Irrigation Canal	
Inside Dimensions	2 m wide x 2 m high (The same as existing one)
Length	10 m
Construction Material	Wet Masonry

2) Crossings of Sewerage Pipe beneath Riverbed

There are three sites where existing sewerage pipes of Addis Ababa Water and Sewerage Authority (AAWSA) cross riverbed, namely, two sites near the National Hotel along the Bantyiketu river and a site immediately upstream from the Filwiha bridge, at the confluence of the Kechene river and the Kurtume river. Among the three sites, the two crossings near the National Hotel along the Bantyiketu river have to be reconstructed because riverbed at the sites is to be excavated down to design riverbed determined by this study.

Like the existing crossings, the new sewerage pipes are to be wrapped with reinforced concrete for protection, where they cross the riverbed.

6.3.2 Bantyiketu and Kostre Regulating Ponds

The proposed Bantyiketu regulating pond is to be built at the left bank, immediately downstream from the Filwiha bridge along the Yohanis street, amid a busy downtown area. The foundation rock at the proposed site of the regulating pond is considered to be basalt, the same as the rock outcropped at the adjacent riverbed and its depth below ground surface is about 3 m, lying as deep as the outcropping rock at the riverbed. A sewerage pipe of Addis Ababa Water and Sewerage Authority (AAWSA) underlying the proposed site is to be relocated.

The proposed Kostre regulating pond is to be constructed at the left bank of the Kostre river, a tributary of the Kechene river, immediately downstream from a bridge along the Dejazmach Haile Silase street. Geologically, the foundation rock at the proposed site of the regulating pond is considered to be basalt, the same as the rock outcropped at the adjacent riverbed and its depth below ground surface is about 5m, lying as deep as the outcropping rock at the riverbed.

The regulating ponds are constructed by excavation of the ground in riparian areas. Along the riverside of the pond, a side overflow dyke and an outlet facility are built. The side overflow dyke is built by excavation and reshaping of the present ground and armoring by wet masonry along its surface. In order to drain impounded water to the outlet facility, drain ditches are constructed along the rim of the bottom of the pond.

The outlet facility of the regulating ponds is made of wet masonry, furnished with a flap gate. The flap gate is selected for fulfilling unmanned operation. Floods are designed to flow into the pond through the side overflow dyke and is returned through the flap gate to the river when the river water has receded. The flap gate made of stainless steel is proposed from the aspect of maintenance-free policy.

For the purpose of avoiding scouring, gabion mattresses are placed on both the riverbed and the pond's bed next to the side overflow dyke as well as the riverbed adjacent to the outlet facility.

Dimensions obtained from the hydraulic analysis in Chapter 4 are finalized by scrutinizing topographic maps from structural aspects. The major features of the two ponds are tabulated as follows.

		Bantyiketu pond	Kostre pond
Reservoir storage volume		73,000 m^3	26,000 m^3
Surface area at highest water stage		29,900 m^2	6,500 m^2
Bottom elevation of pond		2338.5 m	2496.0 m
Side overflow dyke	Crest elevation	2341.8 m	2500.5 m
	Crest length	50 m	30 m
	Height above bottom of pond	3.3 m	4.5 m
	Height above riverbed	4.8 m	3.5 m

The plans of both the Kostre and the Bantyiketu regulating ponds are shown on Figure 6.3.5 and Figure 6.3.6, respectively.

6.3.3 Kechene Weir

(1) Weir Type

The type of weir is determined from topography, geology, size of spillway, availability of construction materials. When the height of dam exceeds 30 m, earth fill weirs shall not be applied.

Major types of weir are as follows.

Concrete Weir	Concrete Gravity Weir
	Concrete Arch Weir
	Concrete Buttress Weir
Fill Weir	Earth Fill Weir
	Rock Fill Weir

1) Viewpoints of Topography

Among various types of concrete weirs, concrete gravity weirs are least influenced by topography. Where a proposed sites forms a U-shaped valley with a large width, the higher the weir height, the higher the feasibility of concrete buttress weirs. When the height of weirs is 80 to 100 m, the weir volume of concrete buttress weirs is generally as much as 70 to 80 % of weir volume of concrete gravity weirs. The smaller the width between abutments at proposed sites, the more feasible the concrete arch weirs. Where the width between abutments at weir crest are 3/2 times as large as the height of weir, the weir volume of concrete arch weirs is about 30 % of that of concrete gravity weirs. Where not only foundations but also abutments are geologically of hard rock and the width between abutments at the weir crest are within three times as large as the height of weir, concrete arch weirs are generally less costly than concrete gravity weirs.

As fill weirs are not allowed to have a spillway in their embankment and, instead, have to have it directly built on sound rock at abutments, their weir volume is much more than the weir volume of concrete weirs.

In addition, the narrower the mountain ridges where weir abutments are thrusted, the higher the possibility of the permeability and weathering at ridges.

2) Viewpoints of Geology

a) Concrete gravity weir

For concrete gravity weirs, foundation rock is required to have sufficient shear strength and faults often become a critical factor.

b) Concrete buttress weir

As concrete buttress weirs have a larger area at the bottom of weir, foundation rock is not expected to have as high shear strength as for concrete gravity weirs. However, where thick river deposits exist, concrete buttress weirs become more costly for the large area at the bottom.

c) Concrete arch weir

Concrete arch weirs require less shear strength of foundation rock than concrete gravity weirs. But both abutments have to be hard and sound rock to withstand arch thrusting force.

d) Fill weir

Foundations at an impervious zone of fill weirs require a higher shear strength and imperviousness and foundations at a pervious zone require a higher shear strength and invulnerability against piping. However, because of a larger bottom area of fill weirs, they require less shear strength of foundations than concrete weirs. Soils foundations are not suitable for high fill weirs because of their small shear strength and high susceptibility for sliding and settlement.

In this study, the type of the proposed weir is concrete gravity. The reasons are:

- Topographically, the proposed site does not form a narrow gorge. This means that concrete arch weirs are inapplicable.
- Geologically, foundations are of hard and sound rock, but both abutments are of weathered rock. This means that concrete arch weirs are inapplicable and that both fill weirs and concrete gravity weirs applicable. Further, the volume of concrete gravity weirs are much less than that of fill weirs, the former being less costly than the latter.
- Construction materials for fill weirs are not abundantly available in the vicinity of the proposed site.

3) Size of Spillway

When the magnitude of floods related to the design of weirs is large, it has a great influence on determination of the size of a spillway and hence the type of dam. A spillway shall be made of concrete because it has to pass rapid flood flows safely. As a spillway of fill weirs has to be built on hard rock at abutments, separate from an embankment, the construction cost of a spillway of fill weirs tends to be occupy a larger portion of total construction cost of a weir than concrete weirs.

(2) Crest Elevation of Non-overflow Section of Weir

Two kinds of spillways are furnished at the proposed weir, namely, an emergency spillway of overflowing type located at the weir's crest and a non-emergency spillway of hollow type at the weir's lower portion. Both spillways are not equipped with gates because gate operation during floods is misleading when the catchment area of the weir is as small as 5 km^2. With the emergency spillway ungated, the crest elevation of non-overflow section of a weir is to be the highest elevation of the following two reservoir water stages as stipulated in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan.

$Hn + hw + he$ (If $hw + he < 2$, then $Hn + 2$)
$Hs + hw + he/2$ (If $hw + he/2 < 2$, then $Hs + 2$)
$Hd + hw$ ($hw < 1$, then $Hd + 1$)
where

Hn : Normal High Water Stage. This is defined as the highest water stage in non-flood seasons.

Hs : Surcharge Water Stage. This is defined as the highest water stage in flood seasons.

Hd : Design Water Stage. This is defined as the highest water stage when the weir design flood flows down the spillway. Here, the weir design flood of concrete weirs is defined as the largest flood among the following three.

① Probable 200-year flood at proposed weir sites

② Maximum previous flood at proposed weir sites

③ Maximum previous flood at sites similar to proposed dam sites with respect to hydrology and climate; Creager's curve is customarily used.

According to the hydrological study, probable 200-year flood at the proposed weir site, which is 120 m^3/s, is the largest among the said three kinds of flood discharge.

hw : The height of wave caused by wind at Hd (Weir Design Water Stage)

he : The height of wave caused by an earthquake at Hn (Normal High Water Stage)

As the reservoir of the proposed weir is impounded with water as short as less than three hours during floods, the proposed weir is not furnished with the water stage defined above as Normal High Water Stage (Hn). Hence, the crest elevation of non-overflow section of the proposed weir is determined as whichever larger between the second and third formulas among the three described above, which are as follows.

$Hs + hw + he/2$ (If $hw + he/2 < 2$, then $Hs + 2$)
$Hd + hw$ ($hw < 1$, then $Hd + 1$)

Surcharge Water Stage (Hs) equals the crest elevation of overflow section of the spillway, which is 2509.0 m. Based on Surcharge Water Stage (Hs), the crest elevation of non-overflow section is derived as follows.

Crest elevation of non-overflow section of the proposed weir	=	Hs + 2
	=	(Crest elevation of overflow section of spillway) + 2
	=	2,509.0 + 2.0
	=	2,511.0 m

On the other hand, Design Water Stage (Hd) is derived from the following formula in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan, which is to obtain the overflow width of spillways of Sabo dams, given as below.

$Q = (0.71\ h_3 + 1.77\ B_1)\ h_3^{3/2}$

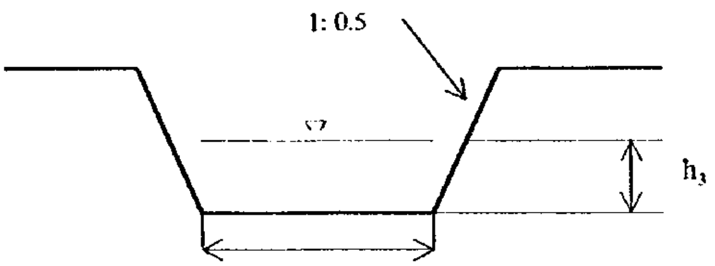
where,

Q : Probable 200-year flood or120 m^3/s (m^3/s)

h_3 : Overflow depth at spillway (m)

B_1 : Width at the bottom of overflow section of spillway (m);

Here, B_1 is assumed as 20 m.



B_1 = 20 m

Substituting Q = 120 m^3/s and B_1 = 20 m into the formula above, we obtain h_3 = 2.2 m. For a conservative estimate, h_3 is made equal to 2.5 m. Hence, the crest elevation of non-overflow section of the proposed weir is obtained as follows, based on Design Water Stage (Hd).

Crest elevation of non-overflow section of the proposed weir	=	Hd + 1
	=	(Crest elevation of overflow section) + h_3 + 1
	=	2,509.0 + 2.5 + 1
	=	2,512.5 m

The two kinds of the crest elevation of non-overflow section of the proposed weir are summarized as follows.

Formula Applied	Crest elevation of non-overflow section
Hs (Surcharge Water Stage) + 2	2,511.0 m
Hd (Design Water Stage) + 1	2,512.5 m

From the table, the crest elevation of non-overflow section of the proposed weir is determined as 2,512.5 m.

(3) Requirements for Stability Analysis of Concrete Gravity Weirs

According to Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan, requirements for stability analysis of concrete gravity weirs are tabulated as follows.

Assumption of Weir Body as a Solid Model	Requirement for Stability Analysis
Two dimensional elastic object	1) Middle Third Condition A composite force shall act within the middle third of the bottom of a weir in a direction perpendicular to the axis of a weir. 2) Safety against shear at the bottom of a weir Safety factor of shear at the bottom of a weir shall be at least 4.0 by using the Henny's formula.

(4) Loading Conditions Required for Stability Analysis of Concrete Gravity Weirs

According to Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan, loading conditions required for the stability analysis of concrete gravity weirs are determined as follows, which are classified by reservoir water stages.

Reservoir Water Stage	Loads
Surcharge Water Stage (Hs)	1) Weir's own weight 2) Static water pressure 3) Seismic water pressure due to reservoir water 4) Sediment force due to reservoir sedimentation 5) Seismic force due to weir's own weight 6) Uplift at the bottom of a weir due to reservoir water
Design Water Stage (Hd)	1) Weir's own weight 2) Static water pressure 3) Sediment force due to reservoir sedimentation 4) Uplift at the bottom of a weir due to reservoir water
Without Impounded Water	1) Weir's own weight 2) Seismic force due to weir's own weight

(5) Estimation of Loads

Various kinds of load are estimated in manners as provided in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan, which are shown as follows.

1) Weir's Own Weight

Weir's own weight is calculated based on the unit weight of constructions materials. In this study, the unit weight of the concrete gravity weir is obtained from EBCS-1, Ethiopian Building Code Standard (1995 edition), Basis of Design and Actions on Structures , published by Ministry of Works and Urban Development, which is 24 kN/m^3 or equivalent to 2.4 tf/m^3.

2) Static Water Pressure

Static water pressure acting upon both upstream and downstream faces of a weir is applied. In the calculation of static water pressure on a weir's upstream face, the height of wave caused by wind and/or earthquake is omitted because it is considered negligibly small. Static water pressure is estimated by the following equation.

$P = Wo\ h$

where

P : Static water pressure (tf/m^2)

Wo : Unit weight of water (tf/m^3)

h : Water depth

According to the said EBCS-1, Ethiopian Building Code Standard, the unit weight of water is $10kN/m^3$ or equivalent to $1.0\ tf/m^3$.

3) Sediment Pressure due to Reservoir Sedimentation

Vertical pressure due to sediment in a reservoir is to equal the submerged weight of sediment. Horizontal sediment pressure at an arbitrary depth shall be calculated by the following equation.

$$Pe = Ce\ W_1 d$$

where

Pe : Horizontal sediment pressure at an arbitrary depth (tf/m^2)

Ce : Coefficient of sediment pressure (0.4 ~ 0.6)

W_1 : Unit weight of submerged sediment (tf/m^3)

W_1 is derived from the following equation.

$$W_1 = W - (1 - \nu)\ W_0$$

where,

W : Wet unit weight of sediment (1.5 ~ $1.8tf/m^3$)

ν : Void ratio of sediment (0.4 ~ 0.6)

W_0 : Unit weight of water ($1.0tf/m^3$)

d : Water depth (m)

4) Uplift at the Bottom of Dam due to Reservoir Water

For concrete weirs without drainage hole at their bottom, uplift is to act vertically upward at the bottom of a weir and its magnitude is to be given as tabulated below, changing linearly between the upstream end of the bottom of a weir and the downstream end. Uplift is not influenced by the fluctuation of reservoir water stages with short duration such as caused by wave.

Location	Magnitude of Uplift
Upstream end at the bottom of weir	(Uplift at the downstream end) + [(Uplift at the upstream end) - (Uplift at the downstream end)]/3
Downstream end at the bottom of weir	Equal to static water pressure adjacent to the downstream end

5) Seismic Force due to Dam's Own Weight

Seismic force due to weir's own weight is to act only horizontally and be given by the following equation.

$I = W k$

where

- I : Seismic force due to weir's own weight (tf)
- W : Weir's own weight (tf)
- k : Seismic Coefficient (=0.10)

  Seismic coefficient applied is calculated by applying factors of seismic zone, subsoil condition and structural ductility, described in EBCS-8, Ethiopian Building Code Standard, Design of Structures for Earthquake Resistance, published by Ministry of Works and Urban Development in 1995. (See Sec.6.2 (2) for details)

6) Seismic Water Pressure due to Reservoir Water

Without relevant experiments, seismic water pressure horizontally acting on the upstream face of a weir is derived by using the Westergaard's formula.

Seismic water pressure at an arbitrary depth is calculated by the following Westergaard's formula.

$pd = 0.875\, W_0\, k\, \sqrt{H h}$

where,

- pd : Seismic water pressure at an arbitrary depth (tf/m^2)
- W_0 : Unit weight of water ($1.0tf/m^3$)
- k : Seismic coefficient
- H : Depth of the bottom of a weir below reservoir surface (m)
- h : Depth where seismic water pressure acts below reservoir surface (m)

Total seismic water pressure is an integral of seismic water pressure at an arbitrary depth (Pd) over entire depth which is given by:

$$Pd = (7/12) W_0 k H^2$$

and Pd acts at a depth given by:

$$Hd = 0.4 H$$

(6) Cross-sectional Dimensions of Proposed Weir

From the middle third condition and the safety against shear at the bottom of a weir, cross-sectional dimensions of the proposed weir are determined as follows, applying the two most critical loading conditions when the reservoir is at Design Water Stage (Hd).

Crest Elevation	Non-Overflow Section	2,512.5 m
	Overflow Section	2,509.0 m
Crest Width	Non-Overflow Section	2.0 m
	Overflow Section	2.0 m
Lowest Elevation of Weir		2,493.0 m
Maximum Height of Weir	Non-Overflow Section	19.5 m
	Overflow Section	16.0 m
Slope of Upstream Face	Between EL. 2,512.5 m and EL. 2,509.0 m	Vertical
	Below EL. 2,509.0 m	1 (Vertical) to 0.2 (Horizontal)
Slope of Downstream Face	Between EL. 2,512.5 m and EL. 2,509.0 m	Vertical
	Below EL. 2,509.0 m	1 (Vertical) to 0.7 (Horizontal)

(7) Design of Emergency Spillway

According to Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan, the design flood of spillways is determined as follows.

For overflow section and shootway	the largest among a probable 100-year flood and a basic flood at a proposed weir site
For stilling basin	the largest among a probable 100-year flood and a design flood regulated by weir

For the proposed weir, flood discharges above-mentioned are tabulated and design floods of the spillway are determined as follows.

For overflow section and shootway	Probable 100-year flood	=	110 m^3/s	Adopted
	Basic flood (Probable 30-year)	=	90 m^3/s	
For stilling basin	Probable 100-year flood	=	110 m^3/s	Adopted
	Design flood Regulated by Weir	=	50 m^3/s	

As catchment area of the proposed weir is less than 20 km^2, the emergency spillway is not furnished with gates as stipulated in Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan

The length of a stilling basin in the direction parallel to flow is obtained from the following formula shown in the Sabo dams' standard in Design Standard of River and Sabo Structures (September 1997 edition) published by Ministry of Construction, Government of Japan.

$L = (1.5 \sim 2.0)(H_1 + h_3)$

where

L : Length of a stilling basin, or distance between the downstream end of the crest of a main weir and the downstream end of the crest of a sub weir (m)

H_1 : Height of the crest of main weir's overflow section above the top of floor slab of a stilling basin(m)

h_3 : Overflow depth at the crest of overflow section of a main weir for probable 100-year flood of 110 m^3/s (m)

Substituting H_1 = 14.5 m and h_3 = 2.1 m to the formula above, L is estimated as follows.

$$\begin{aligned} L &= (1.5 \sim 2.0)(H_1 + h_3) \\ &= (1.5 \sim 2.0)(14.5 + 2.1) \\ &= 25 \sim 33 \text{ m} \\ &= 30 \text{ m as an average} \end{aligned}$$

The height of a sub weir is calculated by the following formula shown in the said Sabo dam's standard.

$H_2 = [(1/3) \sim (1/4)]\, H_1$

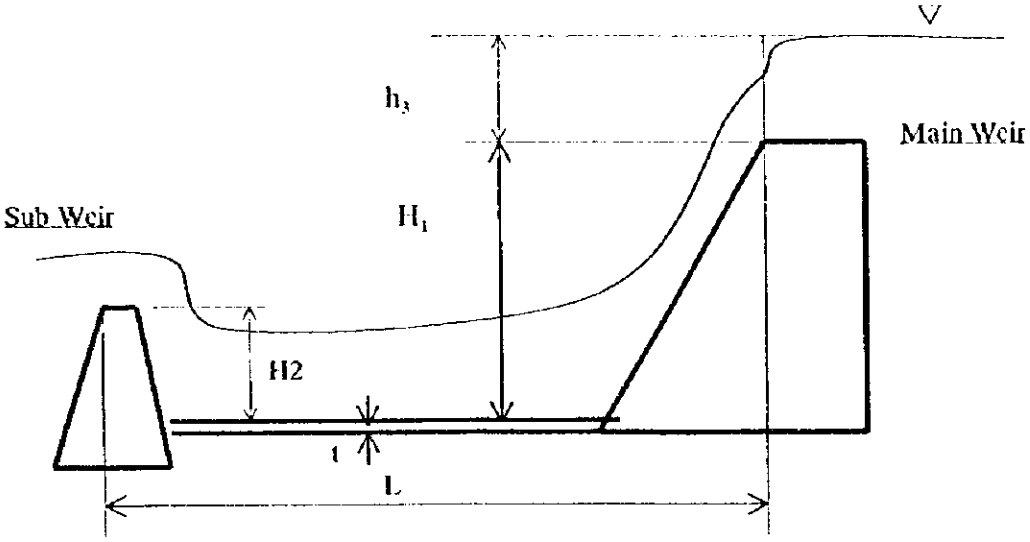
where

H_2 : Height of a sub weir (m)

H_1 : Height of the crest of main weir's overflow section above the top of floor slab of a stilling basin(m)

Substituting H_1 = 14.5 m to the formula above, H_2 is estimated as follows.

$$
\begin{aligned}
H_2 &= [(1/3) \sim (1/4)]\,H_1 \\
&= [(1/3) \sim (1/4)]\,14.5 \\
&= 3.6 \sim 4.8\ \mathrm{m} \\
&= 4.0\ \mathrm{m\ as\ an\ average}
\end{aligned}
$$



The thickness of floor slab of a stilling basin is calculated by the following formula shown in the said Sabo dam's standard.

$t = 0.1(0.6H_1 + 3h_3 - 1.0)$
where

t : Thickness of floor slab of a stilling basin

H_1 : Height of the crest of main weir's overflow section above the top of floor slab of a stilling basin (m)

h_3 : Overflow depth at the crest of overflow section of a main weir weir for probable 100-year flood of 110 m^3/s (m)

Substituting H_1 = 14.5 m and h_3 = 2.1 m to the formula above, t is estimated as follows.

$$
\begin{aligned}
t &= 0.1(0.6H_1 + 3h_3 - 1.0) \\
&= 0.1(0.6 \times 14.5 + 3 \times 2.1 - 1.0) \\
&= 1.5\ \mathrm{m}
\end{aligned}
$$

(8) Major Features of Proposed Weir

The proposed Kechene weir has 5 km^2 of catchment area, located in the upstream stretch of the Kechene river, adjacent to a cemetery named Medhane Alem.

Geologically, the proposed weir site is underlain by basalt, without distinct joints/cracks and with sufficient strength to be suitable for foundation of the proposed weir. The basalt is overlain by sandstone at the left abutment and tuff at the right abutment, both of which are highly weathered and jointed and as deep as maximum 5 m from the ground.

The weir is designed based on Design Standard of River and Sabo Structures (September 1997 edition) published by the Ministry of Construction, Government of Japan. The seismic coefficient to determine the slope of both upstream and downstream faces of the weir in stability analysis is derived from EBCS-8, Design of Structures for Earthquake Resistance, Ethiopian Building Code Standard (1995 edition) published by Ministry of Works and Urban Development and is estimated at 0.1.

The proposed weir consists of a main weir, a sub weir and a stilling basin. A sub weir is situated at the downstream end of a stilling basin. The type of both the main weir and the sub weir is concrete gravity. The side walls and floor of the stilling basin is made of reinforcing concrete.

Downstream from the sub weir, gabion mattresses are furnished on the riverbed to prevent scouring and erosion.

To make dam operation unmanned, the emergency spillway at the crest of the main dam and the non-emergency spillway near the bottom of the main weir are ungated. The emergency spillway is designed for probable 100-year flood and spilling takes place only in the event of floods larger than the design flood (probable 30-year flood). The non-emergency spillway consists of three square orifices, the size of 1.2 m x 1.2m each, near the bottom of the weir, is to discharge floods equal to or smaller than the design flood (probable 30-year flood), as well as instream flow required in downstream areas.

In this context, the reservoir remains empty most of the time even during rainy season because of the non-emergency spillway. And the reservoir water rarely rises to as high elevation as the crest of overflow section of the main weir and stays less than two to three hours in such a high water stage even such a rise takes place. The stability of the main weir without impounding water in the reservoir is emphasized in design.

For making it easy for local people to cross the river at the proposed weir site, a bridge of two spans, the length of a span about 10 m, are to be provided at the emergency spillway. In addition, guard rails are to be furnished along the upstream and downstream sides of the weir crest to secure the safety of pedestrians.

Major features of the proposed weir are as follows.

Type		Concrete Gravity
Crest Elevation	Non-overflow Section	2,512.5 m
	Overflow Section	2,509.0 m
Reservoir Regulating Volume	Gross (Between EL.2,493.0 m and EL.2,509.0 m)	96,000 m^3
	Effective (Between EL.2,499.0 m and EL. 2,509. m)	88,000 m^3
Lowest Elevation		2,493.0 m
Maximum Height	Non-overflow Section	19.5 m
	Overflow Section	16.0 m
Crest Length		120 m
Slope of Upstream Face	Between EL.2,512.5 m and EL.2,509.0 m	Vertical
	Below EL.2509.0 m	1 (Vertical) to 0.2 (Horizontal)
Slope of Downstream Face	Between EL.2,513.0 m and EL.2,509.0 m	Vertical
	Below EL.2509.0 m	1 (Vertical) to 0.7 (Horizontal)
Emergency Spillway	Type	Ungated
	Crest Length	20 m
Non-emergency Spillway	Type	Ungated
	Number of Orifices	3
	Threshold Elevation	2,499.0 m
	Dimensions of an Orifice	Square of 1.2 m x 1.2 m

The plan, downstream view and longitudinal profile of the weir are shown on Figure 6.3.7 and Figure 6.3.8. The reservoir storage volume curve is shown on Figure 6.3.9. These figures and the reservoir storage volume curve are depicted on or on the basis of the 1/2,000-topographical maps published in 1995 for Addis Ababa Water Supply

Project Stage III A, as well as a river cross section at the weir axis made by topographic survey.

6.4 Urban Drainage Facilities

For all of seven basins/sub-basins, two kinds of drain ditches are contemplated depending on its purposes and loading conditions as follows.

Purpose	Loading Condition	Structural Descriptions
Trapping of water on road surfaces	Load attributed to automobiles acts	- Top of drain ditches is covered with grating made of galvanized steel
		- Side walls and floor of drain ditches are made of reinforced concrete
Conveying and drainage of trapped water to river	Load attributed to automobiles does not act	- Top of drain ditches is covered with a lid made of reinforced concrete.
		- Side walls and floor of drain ditches are made of unreinforced concrete

Thickness of side walls, floors and lids of drain ditches and the pattern of reinforcing bars, as well as thickness of leveling concrete, are determined on the basis of Standard Design Drawings of Civil Structures No.1 (Drain Ditch, Pipe Culvert, Box Culvert) published by the Ministry of Construction, Government of Japan

Only for Sub-basin E2 where water on road surface is trapped in front of the Finfine National Restaurant along the Yohanis street and is drained out into the Bantyiketu river, a flap gate of stainless steel is to be furnished at its outlet because the design high water stage is higher than the elevation of the outlet.

Typical cross-sections of the two kinds of drain ditches are shown on Figure 6.4.1.

Table 6.2.1 Unit Weight

Materials		Unit weight (kN/m^3)	Remarks
Concrete			
	Light weight	9 - 20	
	Normal weight	24	Density may be in the range 20 - 28 depending on local material
	Heavy weight	> 28	
	Reinforced and prestressed concrete	+ 1	
	Unhardened concrete	+ 1	
Masonry			
	Basalt	27	
	Limestone	25	
	Granite	27	
	Sandstone	23	
	Trachyte	26	
Water	Fresh	10	

Table 6.2.2 Bedrock Acceleration Ratio (α_0)

Zone	4	3	2	1	0
α_0	0.10	0.07	0.05	0.03	0.00

Table 6.2.3 Importance Categories and Importance Factors for Buildings

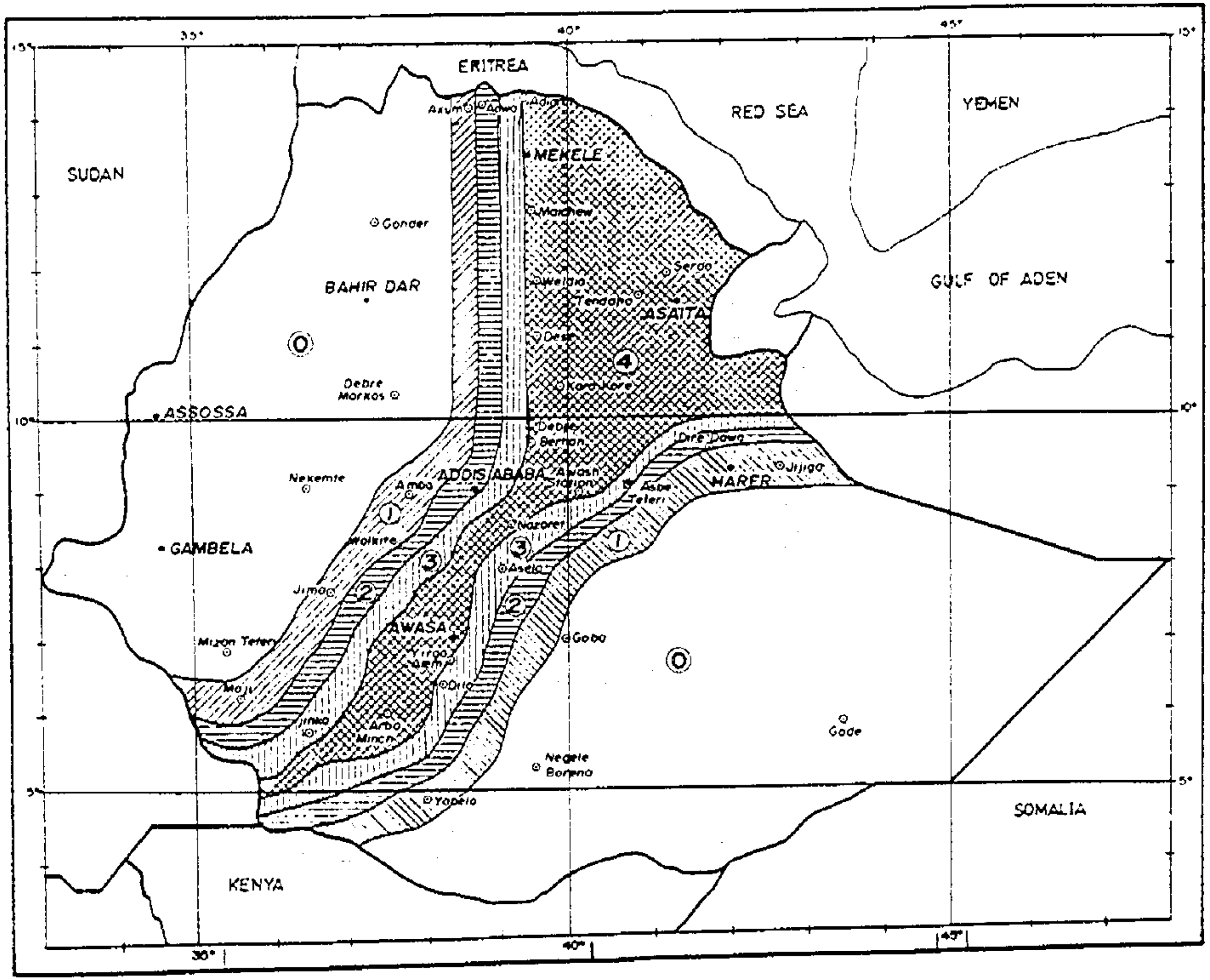
Importance Category	Buildings	Importance Factor I
I	Buildings whose integrity during earthquakes is of vital importance for civil protection ,e.g., hospitals, fire stations, power plants etc.	1.4
II	Buildings whose seismic resistance is of importance in view of the consequences associated with collapse, e.g., schools, assembly halls, cultural institutions, etc.	1.2
III	Ordinary buildings, not belonging to the other categories	1.0
IV	Buildings of minor importance for public safety, e.g., agricultural buildings etc.	0.8

Table 6.2.4 Site Coefficient

Subsoil Class	Subsoil Conditions	Site Coefficient (S)
A	-Rock or other geological formation characterized by a shear wave velocity of at least 800 m/s -Stiff deposits of sand, gravel or over consolidated clay, at least several tens of meters thick, characterized by a shear wave velocity of at least 400 m/s at a depth of 10m	1.0
B	Deep deposits of medium dense, gravel or medium stiff clay with thickness from tens to hundreds meter, characterized by a shear wave velocity of at least 200 m/s at a depth of 10m and at least 350 m/s at a depth of 50 m	1.2
C	-Loose cohesionless soil deposits with or without some soft cohesive layers, characterized by a shear wave velocity below 200 m/s in the uppermost 20 m -Deposits with predominant soft-to-medium stiff cohesive soils, characterized by a shear wave velocity below 200 m/s in the uppermost 20 m	1.5

Table 6.2.5 Reservoir Sediment Volume of Proposed Kechene Weir

Sites of which Suspended Load Measurement was Conducted		Gerbi Dam	Chacha	Robi Gomoro	Berga	Average
Suspended Load	(ton/km²/year)	60.9	101	102.7	64.5	82
Bed Load	(ton/km²/year)	6.09	10.1	10.27	6.45	8
Total Sediment Load in Weight	(ton/km²/year)	66.99	111.1	112.97	70.95	91
Unit Weight	(ton f /m³)	0.91	0.91	0.91	0.91	0.91
Total Sediment Load in Volume	(m³/km²/year)	73.6	122.1	124.1	78.0	99
Trap Efficiency	(%)	100	100	100	100	100
100-year Sediment Volume of Kechene Weir (m³)	(CA=5 km²)	37,000	62,000	63,000	39,000	51,000



(Source : Ethiopian Building Code Standard, EBCS-8, DESIGN OF STRUCTURES FOR EARTHQUAKE RESISTANCE, Ministry of Works and Urban Development, Addis Ababa, Ethiopia, 1995)

Fig. 6.2.1 Seismic Hazard Map of Ethiopia

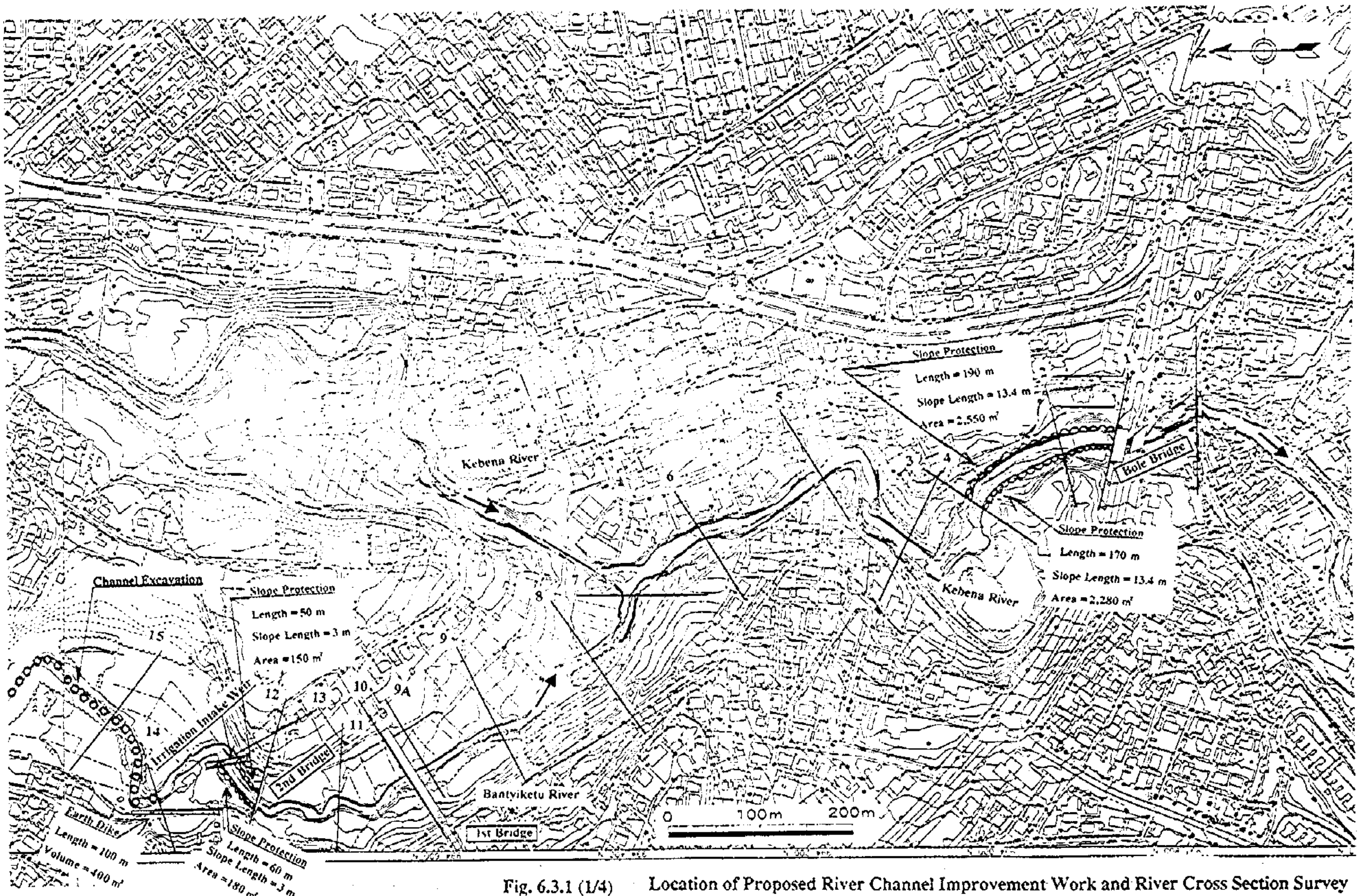


Fig. 6.3.1 (1/4) Location of Proposed River Channel Improvement Work and River Cross Section Survey

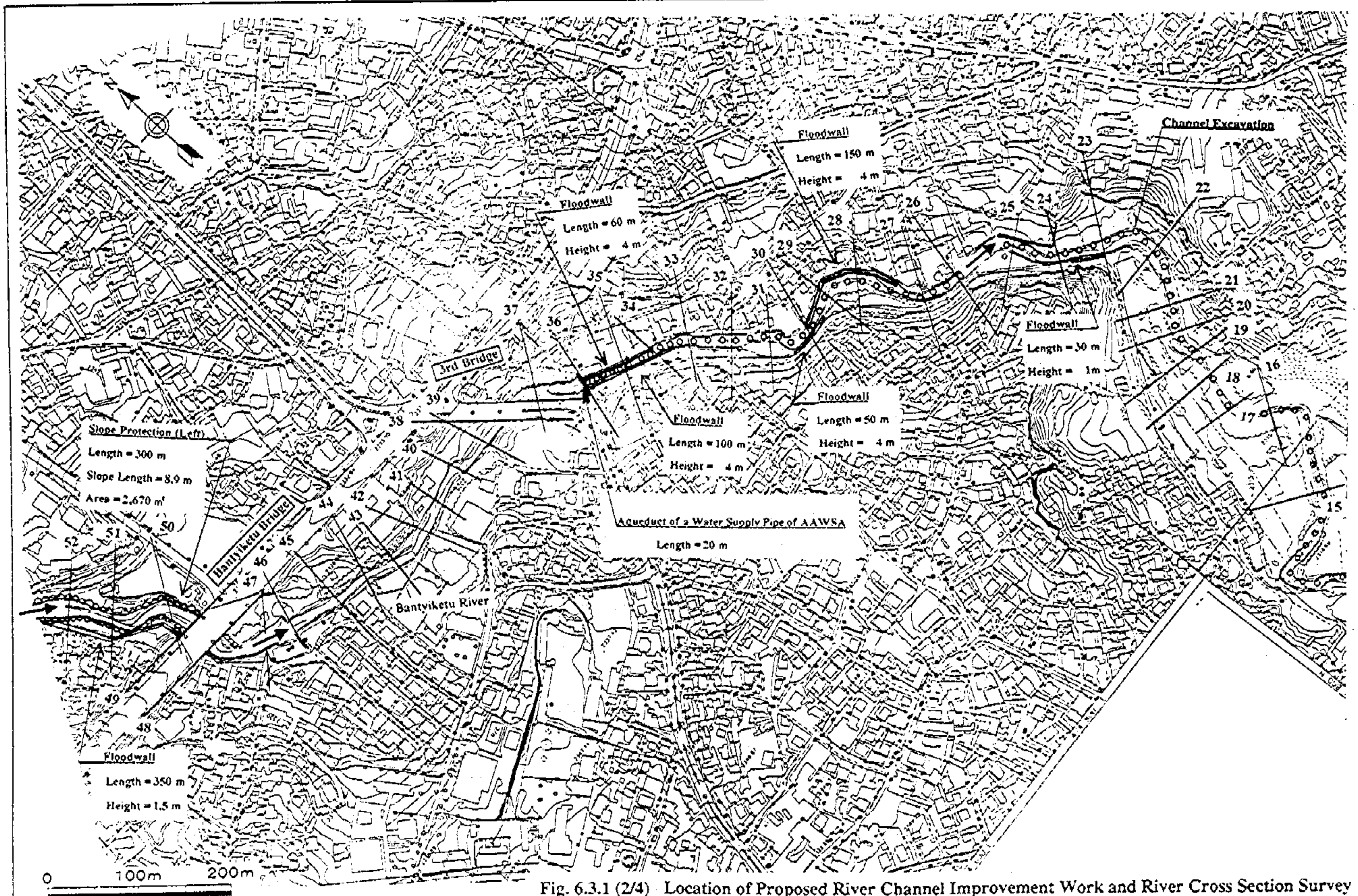


Fig. 6.3.1 (2/4) Location of Proposed River Channel Improvement Work and River Cross Section Survey

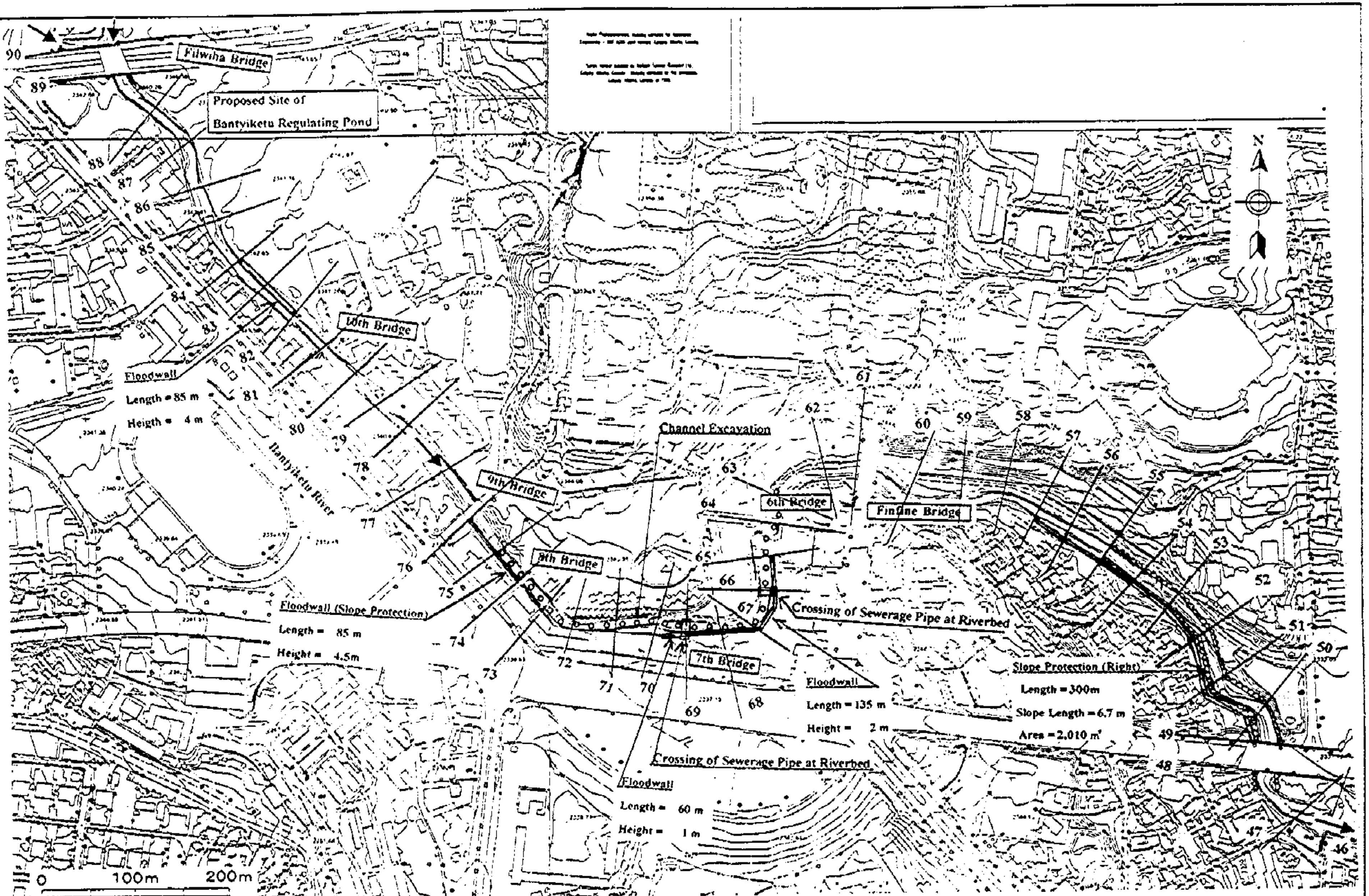


Fig. 6.3.1 (3/4) Location of Proposed River Channel Improvement Work and River Cross Section Survey

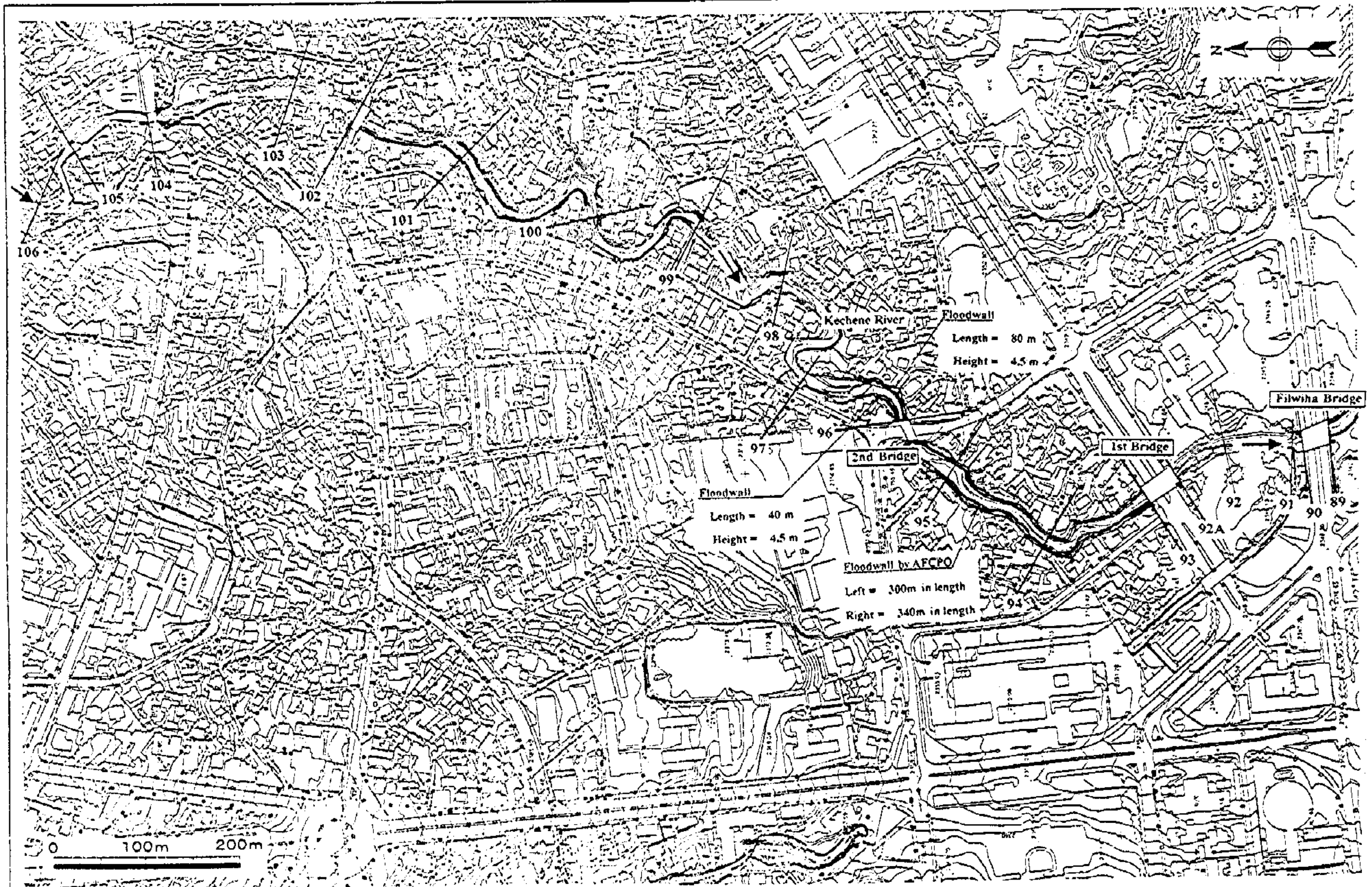


Fig. 6.3.1 (4/4) Location of Proposed River Channel Improvement Work and River Cross Section Survey

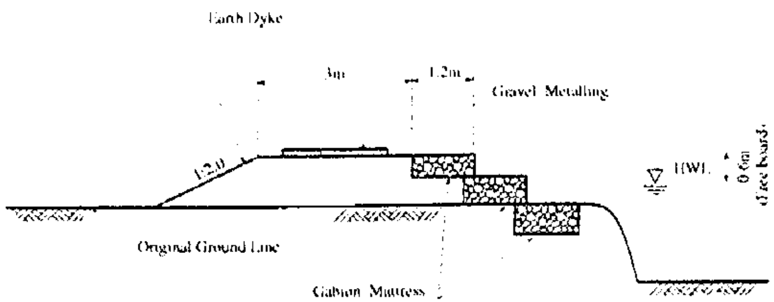


Fig.6.3.2 Typical Cross Section of Earth Dyke

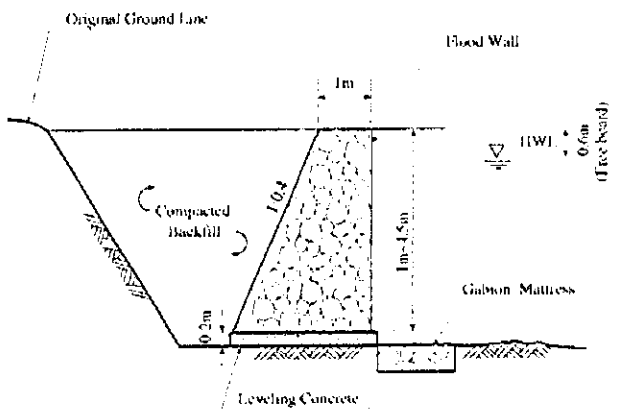
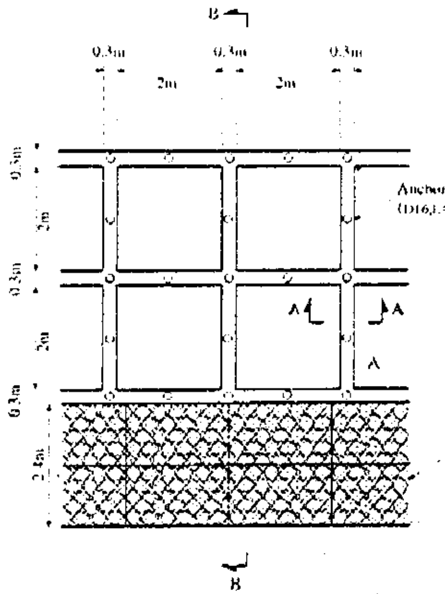
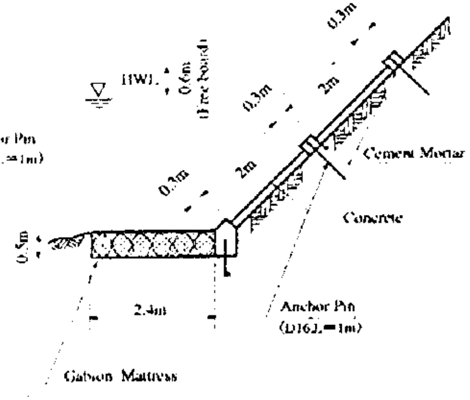


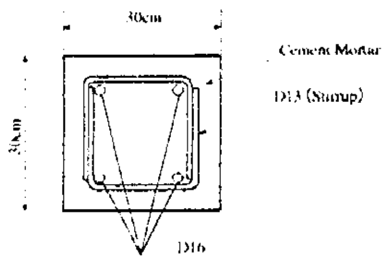
Fig.6.3.3 Typical Cross Section of Flood Wall



Plan



Cross Section
(Section B—B)



Detail Cross Section of Frame
(Section A—A)

Fig.6.3.4 Typical Plan and Cross Section of Slope Protection Work

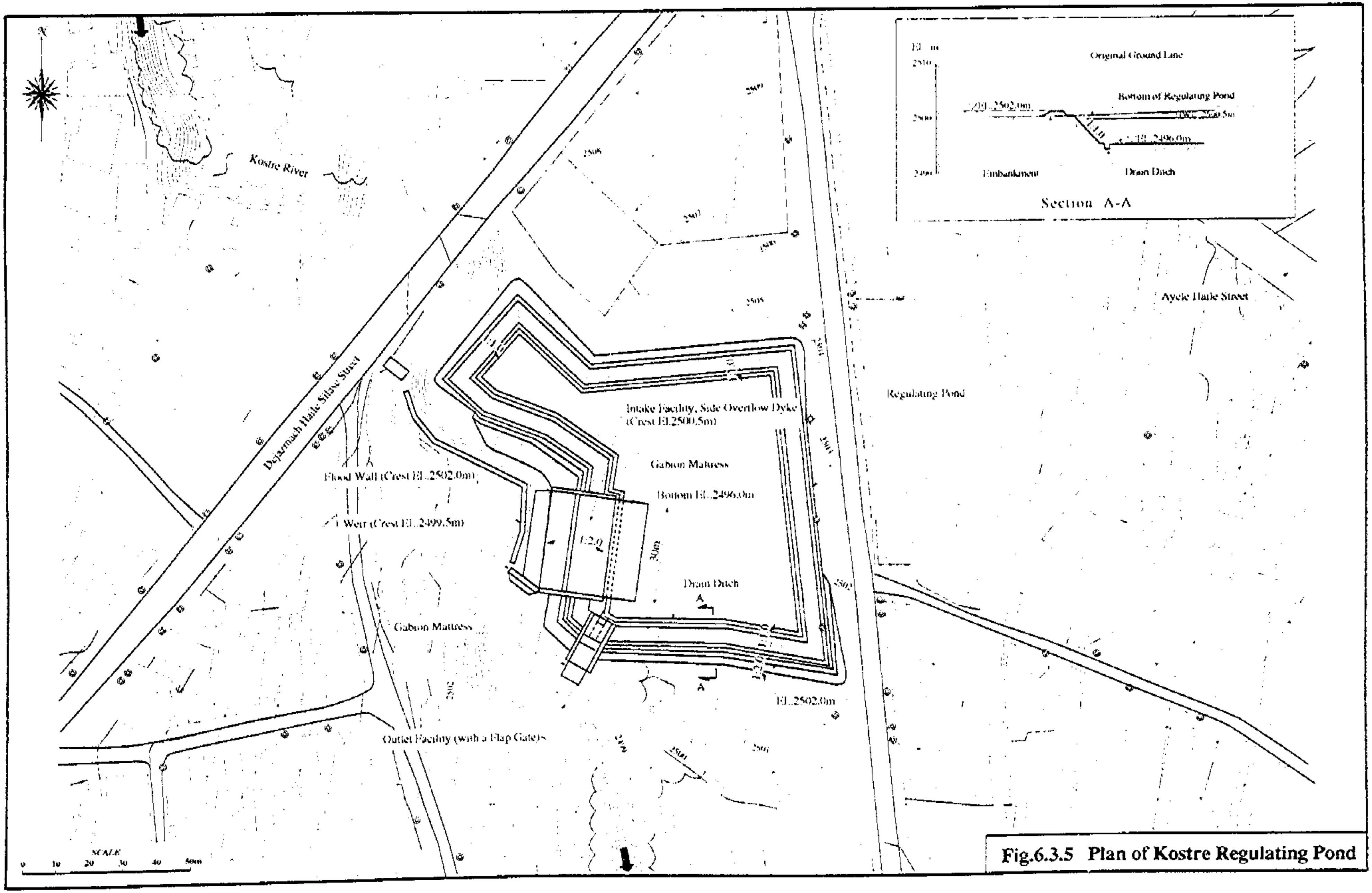


Fig.6.3.5 Plan of Kostre Regulating Pond

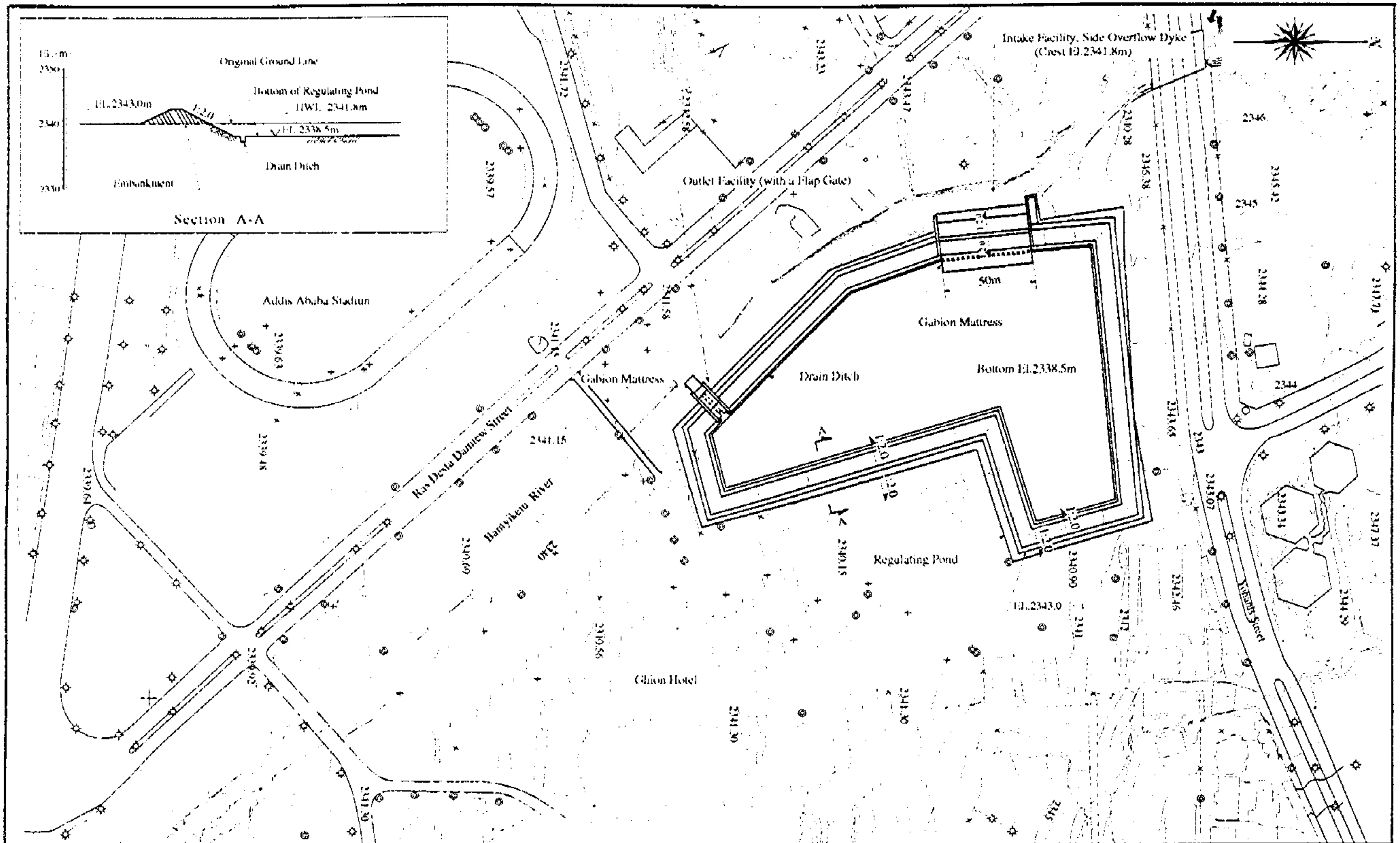


Fig.6.3.6 Plan of Bantyiketu Regulating Pond

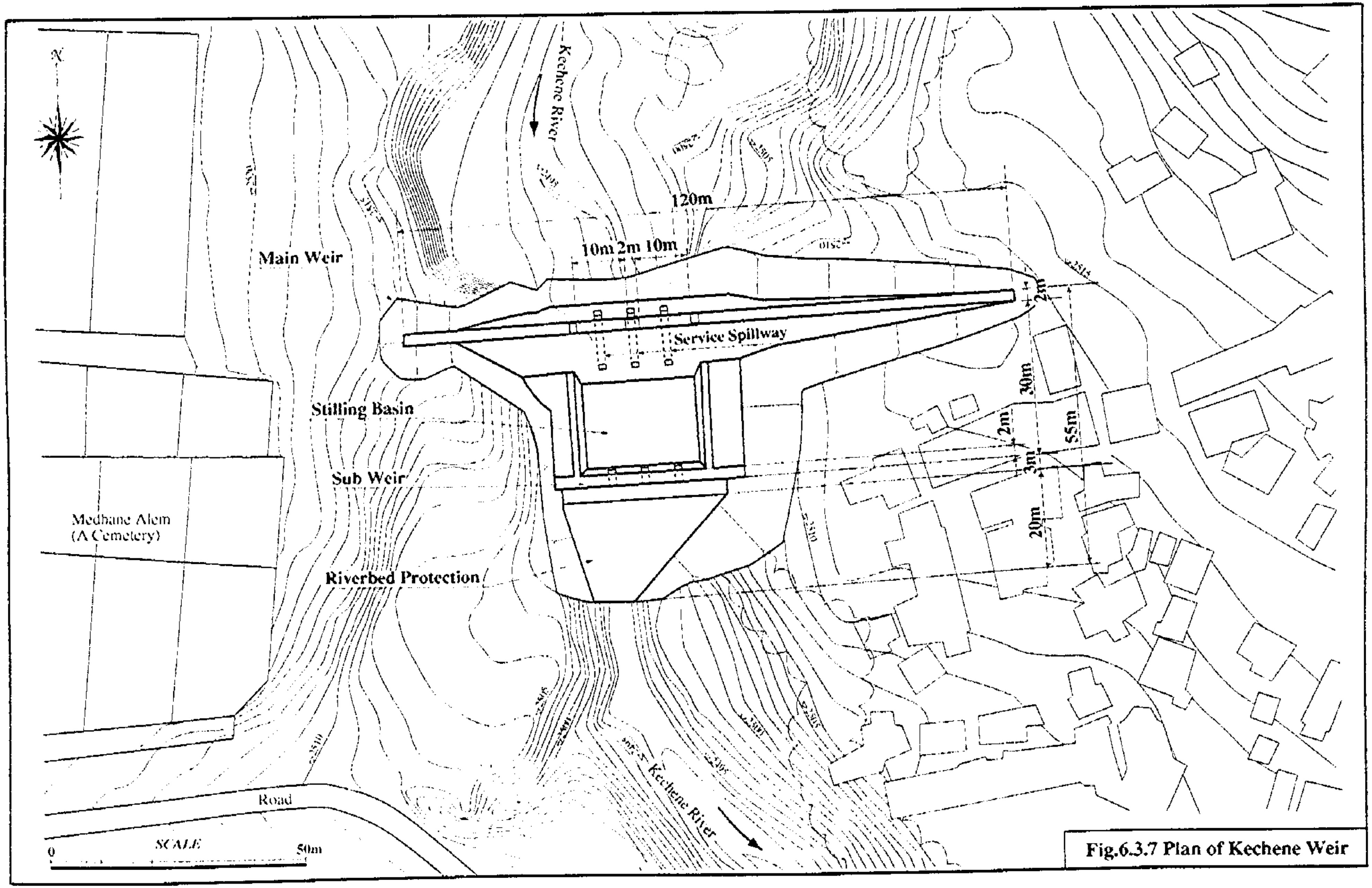
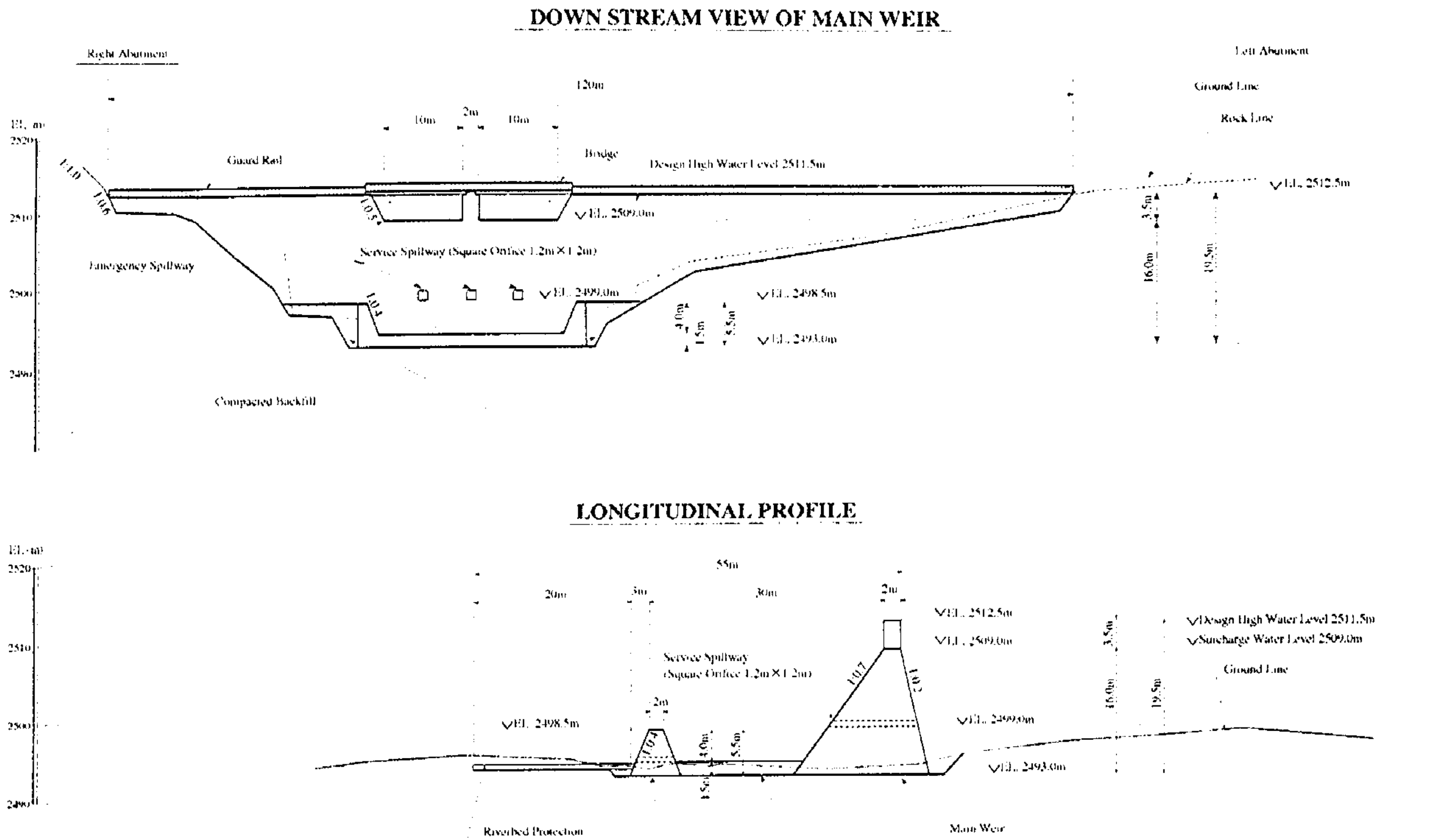


Fig.6.3.7 Plan of Kechene Weir



Note : Consolidation grouting and curtain grouting
are to be applied to foundations
if their necessity is substantiated by
geological investigations.

Fig.6.3.8 Down stream View and Longitudinal Profile of Kechene Weir

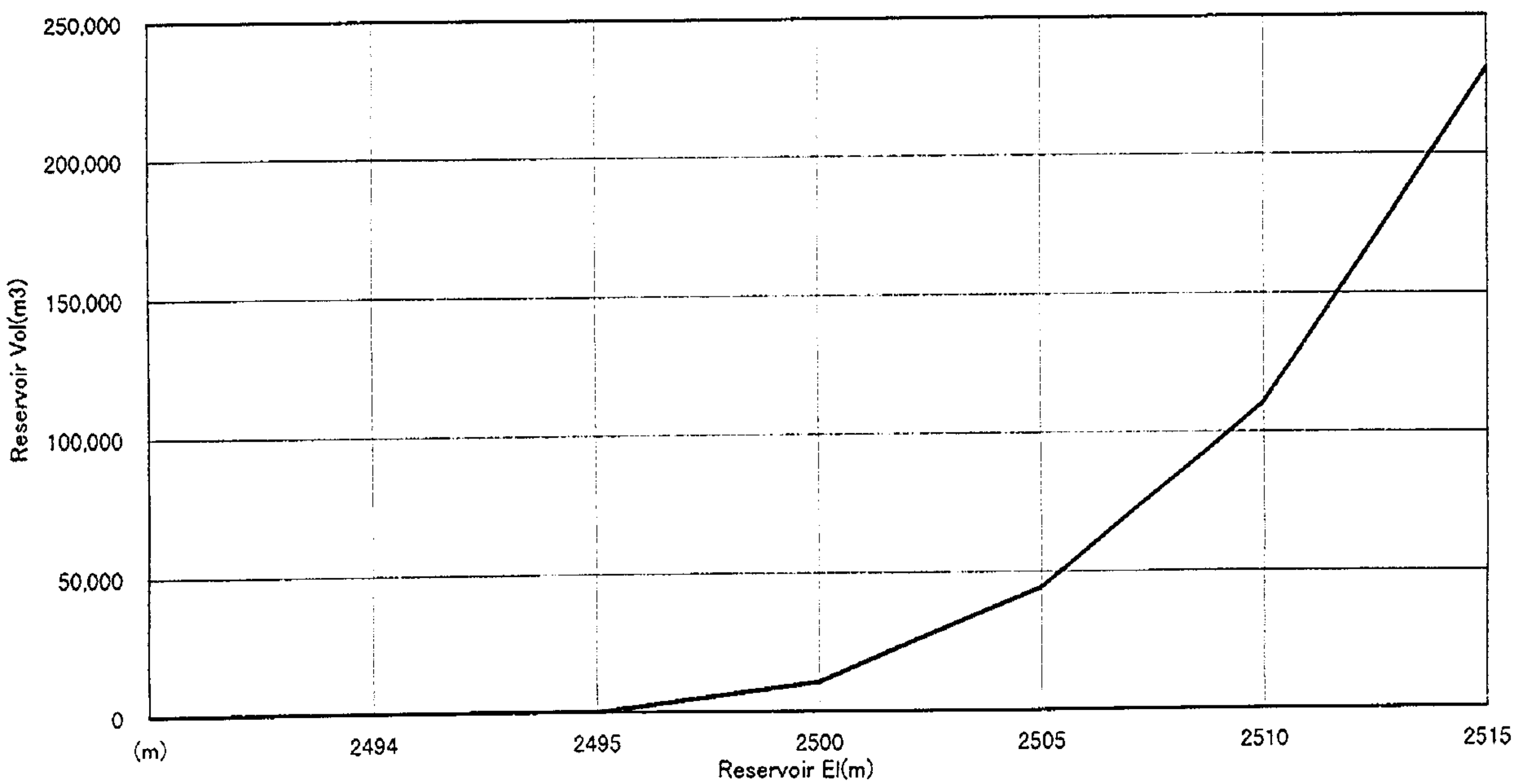
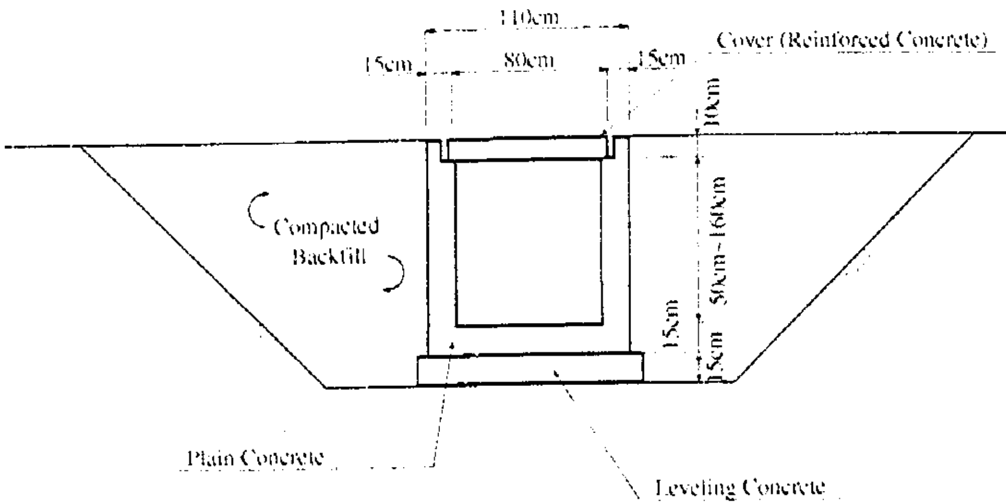
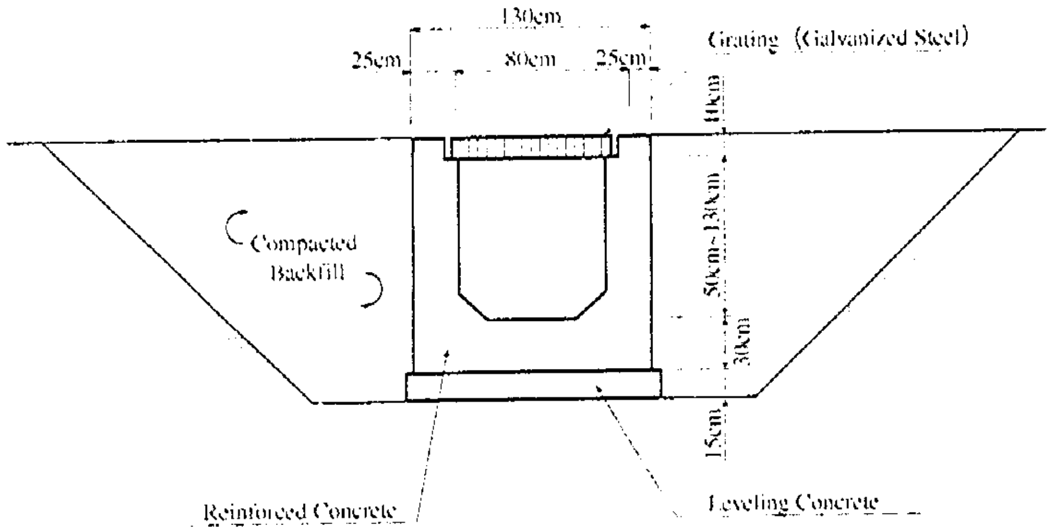


Fig. 6.3.9 Reservoir Storage Volume Curve of Kechene Weir



Standard Drain Ditch



Drain Ditch at Road Crossing

Fig.6.4.1 Typical Cross Sections of Drain Ditches

CHAPTER 7

CONSTRUCTION PLAN AND COST ESTIMATE

THE STUDY

ON

ADDIS ABABA FLOOD CONTROL PROJECT

IN

THE FEDERAL DEMOCRATIC REPUBLIC OF ETHIOPIA

CHAPTER 7 CONSTRUCTION PLAN AND COST ESTIMATE

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7. CONSTRUCTION PLAN AND COST ESTIMATE

7.1 General

The following major assumption and conditions are incorporated in the construction plan and cost estimate.

(1) Procurement

Construction works of project facilities will be executed on contract basis through international competitive bidding, considering the scale of the works. Engineering works of design and supervision will be also executed on international contract basis.

(2) Construction Schedule

Time and duration of construction are scheduled respecting the implementation plan proposed in the master plan study. Construction works are scheduled so that the upper located facilities should be commenced prior to the downstream facilities in principle.

(3) Construction Method

Conventional and common construction methods will be introduced to the project construction works. Employment of human power will be incorporated as much as possible in the project construction works.

(4) Price Level

All the cost is estimated based on the labor wage, the material price and the unit operation cost of equipment as of June 1997.

(5) Project Cost

Initial investment cost for structural measures comprises 1) construction cost, 2) engineering service cost, 3) resettlement cost, 4) administration cost, 5) physical contingency and 6) price contingency. Initial investment cost for non-structural measures comprises 1) installation cost, 2) administration cost, 3) physical contingency and 4) price contingency.

Annual operation and maintenance (O&M) cost comprises those of structural measures and non-structural measures.

7.2 Construction Plan

7.2.1 Material Source

Construction plan is prepared based on the following procurement schedule of major construction materials.

1) Earthfill material will be diverted from excavated surplus material.
2) Concrete will be purchased from a domestic concrete vendor in the Akaki Industrial Estate.
3) Form and reinforcing bar for the concrete will be imported.
4) Quarried stone for wet masonry works will be purchased through local suppliers from the quarries controlled by governmental agencies.
5) Stainless steel flap gate for regulating ponds will be imported.
6) Fuel for construction equipment will be purchased in bulk through import traders.

7.2.2 Transportation

The Djibouti seaport of the Republic of Djibouti will be used for import of goods. For the inland transportation from the seaport to the bonded warehouse in Addis Ababa, which is located beside the Bole international airport, materials and equipment will be transported with lorry and trailer on asphalt paved road. The route of the road is around 950 km long from Djibouti to Addis Ababa through Dobi, Yangudi Rasa, Awash, and Nazret.

7.2.3 Disposal area

Two (2) disposal areas will be provided, 5-10km far from the construction sites, in the suburbs of the city. One is located near to the Bole International Airport (call Bole), and the other is near to the Augusta Garment Factory (call Augusta). Those locations are shown in Figure 7.2.1, and detail of the Augusta Disposal Area is shown in Figure 7.2.2. Each area consists of three (3) to five (5) vacant lots, which are left as old quarries.

It is proposed to dispose the surplus materials to the Bole Disposal Area prior to the Augusta Disposal Area, because the Bole area is remoter from residential area than the Augusta area.

The total capacity of the vacant lots for disposal is estimated at more than 400,000 m3. It is more than the volume of the surplus materials, estimated as approximately 200,000 m3, to be disposed after diverting some excavated materials to filling works. The calculated disposal volume is given in Table 7.2.1.

Usually proposed disposal areas are to be authorized in the following procedure.

1) The Addis Ababa River Management Authority, executing body of construction works, proposes disposal areas to the Unit of Information for City Development, WUDB through Economic Sector, which controls right of ownership and right of use of all the real estate in the Addis Ababa City.

2) The Unit of Information for City Development consults with the mother map, which contains whole the land and property information as they are (present condition) and as they will be (proposed condition on development plan).

3)* The Unit of Information for City Development raises up the issue of the land development plan of the proposed areas with the present data of the proposed areas in a committee of executing committee members of Economic Sector.

4)* The committee decides through a coordination of land development plan among all the authorities and offices in the Region 14 Administration.

5)* The Unit of Information for City Development registers the land development plan of the proposed disposal areas in the mother map.

6) The Unit of Information for City Development informs the registration to the Addis Ababa River Management Authority.

7) The Addis Ababa River Management Authority issues a permission of the disposal area.

Note: The asterisked procedures of 3), 4) and 5) are neglected in the future construction stage, in case the executing body has already apply the location of disposal area to the Unit of Information for City Development.

7.2.4 Workable Days

Workable days of each month for construction works are estimated as follows.

1)	January	24 days
2)	February	23 days
3)	March	24 days
4)	April	22 days
5)	May	22 days
6)	June	24 days
7)	July	23 days
8)	August	23 days
9)	September	22 days
10)	October	26 days
11)	November	25 days
12)	December	27 days
	Total	285 days

The workable days are estimated deducting the number of Sunday and the National holidays and the number of days having rainfall more than 20mm from the number of calendar days in each month. The calculation is given in Table 7.2.2. The number of rainfall days is counted on the daily precipitation record of Addis Ababa for forty six (46) years from 1951 to 1996. There are thirteen (13) National holidays a year in Ethiopia as listed in Table 7.2.3.

7.2.5 Major Work Items

The following works are identified as major work items for the project.

1) Excavation work:	at weir construction site, at pond construction site, at river channel improvement work site and at urban drainage development work site
2) Concrete work:	at weir construction site and at urban drainage development work site
3) Wet masonry work:	at pond construction site, at river channel improvement work site and at urban drainage development work site

7.2.6 Construction Method

The excavation works, at the weir and the regulating ponds sites, will be carried out with bulldozer and hydraulic excavator. The excavation works, at the river channel improvement work site, will be carried out by manpower. Some of excavated earth material will be used for adjacent earthfilling works for the dike of the regulating pond work sites. And some of excavated rock material will be used for wet masonry works. Other surplus earth materials including stripped material will be hauled by dump trucks and adequately filled at the proposed disposal areas.

Ready mixed concrete will be used for the Kechene weir construction works. It is available at the Akaki Industrial Estate. Though the contractor can purchase the concrete from this plan, he is required to mobilize enough numbers of agitator trucks. At the weir construction site, the ready-mixed-concrete will be placed with crawler crane and concrete bucket.

Wet masonry works are mainly employed for wall structure, bridge pier and abutments and some house buildings in Addis Ababa. The stones are usually obtained through local suppliers from the quarries, which are controlled by governmental agencies, located in the suburbs of Addis Ababa. The habitual method, of the procurement of materials and of the masonry works, will be taken for the project construction works.

7.2.7 Preparatory Works

Prior to the permanent structure construction works, preparatory works including temporary works will be finished. At first, common temporary works, such as preparation of contractor's office, workshop, storehouse and access road to disposal area, will be carried out. Temporary works, such as preparation of access road and stockpile, dewatering works and relocation works of obstacles, will be carried out successively at each work site.

The aqueduct located downstream of the 1st bridge of the Bantyiketu river is relocated to secure the required flow area. The relocation work is undertaken as a permanent work in this project.

The relocation of the buried facilities, such as the water supply and sewerage pipe lines crossing the proposed permanent structures, will be specified as temporary works in construction work contract.

Major temporary works at each construction work site are listed in Table 7.2.4.

7.2.8 Equipment Plan

The requirement of construction equipment is estimated as follows, taking work volume, production rate of equipment and implementation schedule proposed in the master plan study into account.

1)	Bulldozer, 21ton	:	3 units
2)	Bulldozer w/ripper, 32ton	:	1 unit
3)	Crawler drill, hyd., 150kg	:	1 unit
4)	Backhoe, 0.6m3	:	5 units
5)	Dump truck, 10ton	:	12 units
6)	Agitator truck, 3.2m3	:	8 units
7)	Crawler crane, 50ton	:	1 unit
8)	Crawler crane, 35ton	:	2 units

Rehabilitation and reinforcement works of the existing public roads will not be required for transportation of the above equipment to access the work sites.

7.2.9 Construction Schedule

Construction works are scheduled to be commenced from the upstream site as follows, since upstream structures can eliminate flood damage in the downstream site during the construction and can also bring the project benefit earlier.

1) Construction works of the Kechene weir
2) Construction works of the Kostre regulating pond
3) Construction works of the Bantyiketu regulating pond
4) Channel improvement works of the Bantyiketu River
5) Urban drainage improvement works

It is supposed that construction work contract will be concluded at the beginning of 2000 and the whole of construction works will be completed at the end of 2001. It will take 2 years to complete the construction. Construction schedule is shown in Figure 7.2.3.

Successively after the contract of construction works, preparatory works, such as preparation of contractor's office and procurement of construction equipment, will be carried out for two months.

Construction works of the Kechene weir will be commenced prior to the other works. It will take about 18 months form the beginning of 2000 to the middle of 2001. Natural river flow, for the beginning eight months, will be discharged with half closure of river for riverbed protection work, sub weir work and left side excavation and concrete works for main weir. For the successive concrete work of right side main weir, the river flow will be discharged through the aqueduct pipe which is tentatively installed in the orifice section of the main weir. Ready-mixed-concrete will be purchased by contractor. The concrete will be placed into the weir body with crawler crane.

The detail of main construction sequence of the Kechene weir is discussed below.

1) Primary access:	Heavy equipment for excavation works access to the weir site prior to other works. The access route is 120m long from the Abera Gizaw Street to the weir axis along the right side bank of the river. (March, 2000)
2) Excavation (1):	Excavation of common soil and rock in the riverbed and in the left side area of the weir is carried out. Some excavated soil and rock materials are diverted to filling work for the successive construction work of the secondary access road. The other surplus materials are carried out and disposed through the primary access route. River stream section is shifted to the right side of riverbed at the weir site with sand bags. (March - April, 2000)
3) Secondary access	The secondary access road is constructed with filling some of diverted materials produced in the above excavation work (1). The route is 500m long from the Abera Gizaw Street through to the north along the right side bank of the river and to the south to the weir axis across the river. At the crossing point of the access road and the river, pipe culvert is installed tentatively to discharge the river flow downstream. (April 2000)
4) Sub weir	Gabion work for riverbed protection and concrete work for sub weir and for stilling basin are carried out in this order. Ready-mixed-concrete is carried into the site through the secondary access road and is placed with crawler crane. An aqueduct pipe is provided tentatively to discharge the river flow downstream. The pipe is installed from the downstream tip of the culvert of the primary access road to the downstream tip of the sub weir. (May 2000)
5) Main weir (1)	Concrete works of the left side main weir body is carried out. The concrete is placed batch by batch into the main weir body. One batch size is 1.5m high, 15m wide and 3 to 16m deep from upstream weir surface to downstream surface depending on the height. Ready-mixed-concrete is placed by crawler crane as same as sub weir. River stream section is shifted with sand bags to the right side of sub weir for discharging river flow downstream at the weir site. (May - October, 2000)

6) Excavation (2) — Excavation of common soil and rock in the right side area of the weir is carried out. The excavated materials are carried out and disposed through the secondary access road. River flow is discharged downstream as same as main weir (1) works. (November, 2000)

7) Main weir (2) — Concrete works of the right side main weir body is carried out. Ready-mixed-concrete is placed batch by batch as same as main weir (1) works. An aqueduct pipe is provided tentatively to discharge river flow downstream as same as sub weir works. The pipe is installed in the orifice section at the weir body site. (December, 2000 - July, 2001)

Construction works of the Kostre regulating pond will be commenced one month later from the commencement of the Kechene weir construction works. It will be finished end of 2000 after 9 months. The excavation work in the pond will be carried out by bulldozer. Embankment work can be carried out simultaneously with the excavation work by bulldozer. Wet masonry and miscellaneous works will be carried out in the latter half of the pond construction period.

Construction works of the Bantyiketu regulating pond will be commenced in the middle of 2000, and will be finished at the end of 2001 after one and half year. The excavation work in the pond will be carried out by bulldozer. Embankment work can be carried out simultaneously with the excavation work by bulldozer. Wet masonry and miscellaneous works will be carried out in the latter half of the pond construction period.

Construction works of the Bantyiketu river channel improvement will be commenced end of 2000 and will be finished at the end of 2001 after a year. The excavation and wet masonry works will be carried out by manpower applying conventional work method in Addis Ababa. Many workers will be assigned to work in river zone. The works will be carried out during dry seasons to ensure the safety and the efficiency of the works.

Construction works of the urban drainage improvement will be commenced at the beginning of 2000 and will be finished in the latter half of 2001 after 9 months. Drainage excavation work and concrete work for the construction of drainage ditches will be carried out simultaneously. The grating for drainage ditch will be fabricated in Addis Ababa.