ANNEX–H

DAM AND RESERVOIR

ANNEX - H

DAM AND RESERVOIR

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ANNEX-H DAM AND RESERVOIR

H.1 INTRODUCTION

This ANNEX presents all the results of planning and designing of dams, including review of the previous feasibility study report prepared by WAPDA, 1982 (Ref. 01).

According to the feasibility study report prepared by WAPDA in 1982, the Mol and Khadeji dams were proposed to construct on the Mol and Khadeji tributaries of the Malir river as shown in Fig. H.1-1. Main objectives of the dams were to augment irrigation and potable water supply, and mitigate flood especially in the downstream of the Malir river.

The main works undertaken in this study were to review all the previous studies and to make its revision, if necessary, mainly based on the data and information from the previous reports, and results of additional investigations made during this study. After completion of the previous study, about one decade has passed, and several projects have been implemented in the study area. Such changes were also taken into account in planning of dams, and reviewing the previous study.

H.2 PREVIOUS STUDY

H.2.1 General

The "Feasibility Study on Water Resources Development Project in Malir River" was carried out by WAPDA in 1982. The results of the previous studies on the Khadeji and Mol dams are summarized in this Chapter.

(1) Selection of Damsite

Selection of damsite(s) was carried out in consideration of irrigation water supply and potable water supply as well as flood control of the Malir river. The following four (4) alternative sites were identified:

Case A : Construction of nine check dams along the Malir river.

Case B : Construction of a single dam at about 2 miles downstream from the confluence of the Khadeji and Mol tributaries.

Case C : Construction of a single dam at slightly upstream of the confluence.

Case D : Construction of two separate dams on the Khadeji and Mol tributaries.

Based on preliminary geological investigations, topographic surveys, and cost comparison, it was concluded that construction of the Khadeji and Mol dams was the most economically and technically feasible. Accordingly, detailed feasibility study was performed for the Khadeji and Mol dams.

(2) Reservoir Storage Capacity

The studies on the required capacities of two reservoirs were carried out on the basis of two different techniques so called in the report, (1) the drought period studies and (2) the ripple method. Taking into account the flood control effect by curtailing flood peak, the required reservoir capacities were finally determined at 54.7 MCM for the Khadeji dam and 50.9 MCM for the Mol dam, which were fixed at the maximum of the topographical limits. The following table shows live storage and flood control storage for respective reservoirs:

Unit: MCM

Storage Volume	Khadeji Dam	Mol Dam
Live storage	32.3	33.2
Flood Control Storage	22.4	17.7
Total	54.7	50.9

(3) Salient Features of Proposed Dams

Salient features of the Khadeji and Mol dams proposed by WAPDA are presented in Table H.2.1 and summarized below:

Item	Unit	Khadeji	Mol
1. Hydrology			
- Catchment area	km²	567	611
- Annual rainfall	mm	217	217
- Flood discharge (PMF)	m³/sec	5,210	3,800
2. Reservoir			
- Dead storage	MCM (El.m)	7.2	7.7
- Live storage	MCM	32.3	33.2
- Flood control storage	MCM	22.4	17.8
- Gross storage	MCM	61.9	58.7
- Max. reservoir area	km²	1.42	0.63
3. Dam			
- Type		Concrete gravity	Homogeneous earthfill
- Maximum height	m	39.0	44.2
- Length of crest	m	381	2,347
- Crest elevation	El. m	168.6	177.2
- Maximum water level	El. m	166.3	174.8
- Normal full water level	El. m	162.6	174.8
4. Spillway			
- Type		Overflow (gated)	Submerged weir
- Gates	Nos. x m x m	5 x 12.2 x 5.1	(ungated)
- Capacity	m³/sec	3,830	3,720

H.2.2 Khadeji Dam

(1) Selection of Damsite

Two damsites were identified on the Khadeji Nadi. The damsite No. 1 is located at about 290 m (960 feet) upstream from Khadeji fall and the damsite No. 2 at about 1,000 m (3,300 feet) downstream of Khadeji fall. However, possibilities of constructing a dam at the site No. 2 were ruled out for the following reasons:

- To design the same storage at both dam sites, the height of dam from river bed at the site No. 2 will be about 9.1 m (30 feet) higher than that at the site No. 1.

- The extra height, about 9.1 m (30 feet), at the site No. 2 will make the dam construction more expensive at this location.

- The extra height of about 9.1 m (30 feet) at the site No. 2 is due to the steep slope of river bed and existence of a fall located at about 4.5 m (15 feet) downstream from the site No. 1.
- As the river runs through a narrow gorge between the sites No. 1 and No. 2, no appreciable storage is available between the two sites.

(2) Selection of Dam Type

Comparative cost studies were made between concrete gravity dam and an earthfill dam, and it was concluded that the concrete gravity dam is cheaper than the earthfill dam for the following reasons:

- In the case of an earthfill dam, a separate spillway is needed for flood routing.
- A spill channel is to be dug to carry a 100-year frequency flood discharge with control structures at its inlet and fall structure at the outfall.
- As there is no saddle available for construction of a spillway, construction of spillway structure and spill channel requires a huge excavation of rock.
- In the case of concrete gravity alternative, no extra spillway structure is required.
- Moreover, there are many advantages of a concrete gravity dam as compared with an earthfill dam.

Therefore concrete gravity type of dam was proposed in the previous studies.

(3) Design of Dambody

(a) Overflow Section

From hydrological studies the crest level of the overflow section was fixed at El. 156.5 m (513.5 feet) with radial gates of size 12.2 m x 6.1 m (40' x 20'). Ogee shape crest at El. 156.5 m for 6.1 m (513.5 for 20 feet) designed head was adopted.

(b) Stability Analysis

- Results of Stability Analysis

Critical Condition		Overflow Section	Non-Overflow Section
- Factor of safety against overturning without earthquake pressure	:	3.19	2.87
- Factor of safety against overturning with earthquake pressure	:	2.42	2.18
- Factor of safety against sliding (shearing) without earthquake effect	:	1.20	1.21
- Factor of safety against sliding (shearing) with earthquake effect	:	0.93	0.94

Remarks : The factor of safety against sliding is less than the minimum of 1.5 both for without and with earthquake pressure. So to make the dam safe against sliding, keys have been provided at the heel and toe.

- Compressive Stress Analysis

Compressive stress analysis was made for non-overflow section both at the heel and toe, and the results obtained were reported as follows:

- Normal stress at the toe : 8.0 kgf/cm^2 (113.66 Psi)
- Normal stress at the heel : 0.4 kgf/cm^2 (5.02 Psi)

Allowable compressive stress for concrete foundation is 28-42 kgf/cm^2 (400 - 600 Psi).

(4) Spillway

The crest level of spillway is at El. 156.5 m (513.5 feet) with 5 radial gates with size of 12.2 m x 6.1 m (40' x 20') and maximum spillway capacity is 3,820 m^3/sec (135,000 cusecs) of the maximum discharge with 1,000-year frequency.

(5) Outlet Works

For release of water from the reservoir into the river downstream of the dam for recharge of groundwater, a circular pipe of 1.8 m (6') diameter with control gate 1.8 m x 1.8 m (6' x 6') at its inlet side was proposed.

(6) River Diversion

From hydrological studies, the 20-year flood discharge was estimated at 1,760 m^3/sec (62,000 cusecs). Among many ways of diverting flood flows during construction, it was concluded that the only feasible way at the Khadeji damsite was multistage diversion over the top of alternate construction blocks.

H.2.3 Mol Dam

(1) Selection of Dam Type

The type of dam invariably depends upon the nature of foundation and the availability of construction materials. Considering the width of the river at the proposed dam axis and investigations of materials available in the vicinity, a modified homogeneous section with a 1.5 m (5 feet) thick horizontal drainage blanket and with a toe drain at the extreme end was proposed. Possibility of a zoned embankment section was not considered as no impervious type of materials are available in the vicinity of dam axis.

(2) Design of Dambody

(a) Crest Level

From hydrological studies, the normal full water level was calculated at El. 170.7 m (560.0 feet), and maximum water level during routing of 1,000-year flood through spillway rises to El. 174.8 m (573.5 feet). The crest of the dam was kept at 2.3 m (7.5 feet) higher than the maximum rise of water level during flood routing through the spillway which is El. 177.1 m (581.0 feet).

(b) Stability Analysis

Results of stability analysis are shown below:

Condition		*Factor of Safety*		*Remarks*
- Just after construction	:	(Case I)	1.477	Downstream
		(Case II)	1.482	Downstream R = 250 feet
		(Case III)	1.491	Downstream R = 254 feet
		(Case IV)	1.545	Downstream R = 272 feet
		(Case V)	1.602	Downstream
- Just after rapid drawdown	:	(Case I)	1.352	Upstream
		(Case II)	1.602	Upstream

The critical Case I, just after construction, was analysed for the following three (3) conditions of earthquake force:

Condition		Factor of Safety
- Only horizontal forces acts towards downstream	:	1.17
- The horizontal forces acts towards downstream and the vertical forces upward	:	1.14
- The horizontal forces acts towards downstream and the vertical forces downward	:	1.20

(c) Amount of Seepage Through the Dam

The coefficient of permeability of the fill material for the Mol dam was 1.6x 10^{-4} cm/sec (5.25x10^{-6} feet/sec). The amount of seepage through the dam was determined at 37 l/sec (1.3 cusecs).

(3) Spillway

The crest level of spillway was designed at El. 170.7 m (560 feet), and maximum spillway capacity was 3,710 m^3/sec (131,000 cusecs) of the maximum discharge with 1,000-year frequency.

(4) Outlet Works

Two sites were selected: one through the right abutment of the main river course and the other a little distance away from the left bank of the main course. A comparative cost study showed that the location of the outlet works for the Mol dam along the left bank is more economical than through the right abutment. The topography and geology of the selected site for outlet works indicate that a tunnel can be constructed at this site. The minimum diameter of tunnel which can be constructed is about 1.8 m (6 feet) and the same was proposed.

(5) River Diversion

River diversion works are usually designed to safely carry 20-year flood frequency discharge. The 20-year flood discharge was estimated at 1,270 m^3/sec (45,000 cusec). Considering the topography of the Mol damsite, the only feasible solution was to contain the river flows in the main channel while the works progress.

H.3 PLANNING OF DAMS

H.3.1 General

The basic data used for the previous studies such as catchment area, elevation-capacity curve, were scrutinized in this study. Moreover, meteorological and hydrological data were also reexamined for the planning of dams as discussed in ANNEX-B. All the results of additional geological and soil mechanical analysis were also incorporated in this study, as stated in ANNEX-C. As discussed in ANNEX-D, dam(s) operation has close relation between the allowable discharge from the dam(s) and groundwater recharge in the basin aquifer. All the above results were utilized in planning and designing of dam(s).

In planning and designing the dams, the following topographic maps, geological maps and hydrological data were utilized:

	Khadeji and Mol Damsites	Source or Refer to
1. Topographic map		
- Catchment area	1/50,000 (15.2 m)	Survey of Pakistan
- Reservoir area	1/7,920 (10 feet = 3.0 m)	WAPDA
- Damsite	1/2,400 (5 feet = 1.5 m)	WAPDA
2. Geological map	ANNEX-C	See ANNEX-C and DRAWINGS
3. Foundation and construction materials	ANNEX-C	
4. Hydrological data	ANNEX-B	

Remarks: Parentheses show counter interval in meter.

H.3.2 Review of Previous Study

Prior to planning and designing of dam(s), the review of the previous study was performed mainly based on the additional data and information such as topographic map, results of additional geological investigations, and soil mechanical analysis. The followings are the main review items, and the details of selection of dam type for the Khadeji dam and the Mol dam and other studies are presented in Chapters H.5 and H.6 respectively.

- Damsites
- Elevation-area and elevation-capacity curve
- Type of dam
- Allowable maximum flood surcharge level
- Flood routing

(1) Damsite and Dam Type

(a) Khadeji dam

The Khadeji damsite proposed by WAPDA is selected based on the comparative study as briefly explained in Subsection H.2.2. The proposed damsite is located at a narrow gorge, and both concrete and fill-type dams are geologically as well as topographically suitable. Moreover construction materials for both dam types are available at adjacent area as described in ANNEX-C. However, the cost of spillway would be a decisive factor in selecting dam type due to topographical conditions. In the WAPDA report, it was concluded that the concrete gravity dam was selected based on the preliminary cost comparison. The concrete gravity dam at the proposed damsite is the best selection at this narrow gorge as shown in Fig. H.5-1.

(b) Mol dam

The proposed Mol damsite is topographically more suitable for a fill dam on a relatively wide valley, moreover a suitable spillway site is available on the left bank as shown in Fig. H.6-1. In the WAPDA report, a homogeneous type fill dam was proposed and designed, taking into account availability of filling materials and easiness of construction.

However, judging from the results of laboratory tests carried out by WAPDA and additional laboratory tests in the present study, it indicates that grain size passing 74μ sieve is about 20-50% at the proposed borrow area as analysed in ANNEX-C, and that impermeability of core materials can be kept under the proper moisture control during construction period. Moreover, a zone type rock-fill dam is adopted for the Mol damsite in consideration of availability of core materials, stability of dam body, as well as economic advantage as described in detail in Section H.6.2.

(2) Elevation-Area and Elevation-Storage Curves

Elevation- area and elevation-storage curves were prepared based on topographic maps with a scale of 1/7,920 (8" = 1 mile) and a contour interval of 1.5 m (5 feet).

Though a little difference between the newly prepared curves and the previous one was recognized, the newly prepared curves shown in Figs. H.3.2 and H.3.4 are adopted after scrutinizing the results.

H.3.3 Topography and Geology

(1) Khadeji Dam

The lowest elevation of existing ground surface is 130 m in the river bed and the highest point of elevation is 220 m on right abutment of dam axis. The damsite valley is 170 to 180 m in width.

Gaj formation is exposed in the damsite area and quaternary deposits distribute covering the Gaj formation, as shown in DRAWINGS. Quaternary deposits consist of loose and calcareously sand, debris and cemented gravels as well as conglomerates. The main rock units of Gaj formation are sandy limestone with crystalline limestone and pelitic limestone.

There are no major structure and geomechanical defects in the foundation beds. Folds, faults and open joints are slightly found. The beds are gently dipping more or less horizontal.

The right abutment is composed of sandy limestone with crystalline and pelitic limestone intercalations. The results of core drilling and permeability test carried out along the abutment (KD-2), show good core recovery and low permeability and indicate that the bed rock is competent and devoid of open joints. Solution cavities in the bore hole are not numerous and neighbor appreciable, but along the exposed bed sizable big cavities along bedding planes can be seen.

The slope of the face of the left abutment is quite gentle. Sandy limestone is the common faces with intercalation of crystalline and pelitic limestone around the site. The solution cavities can be seen in the sandy limestone beds. Results of core drilling suggest that limestone has well developed fractures and weathered solution cavities. The core recovery is also less and water loss is very high in the left abutment. These zones shall require special treatment such as grouting with various cement mixtures.

The valley is 180 m in width and about 8 m thick overburden that is well cemented gravels and also loose river deposits, distributes along the river. The overburden covers on the bed rock in the valley along the abutments. Information obtained from core drilling suggests that limestone group is sufficiently compact and water tight.

(2) Mol Damsite

The river has a relatively wide valley and the slope on both abutments is moderately steep near the proposed dam axis. The river valley is about 100 m wide at the axis. The lowest elevation in the river bed is about 133 m, while the highest elevation at both the abutments is about 175 m.

Quaternary deposits underlain by the Gaj formation are distributed in the area shown in DRAWINGS. Quaternary deposits consist of loose to calcareously cemented debris, gravels and sand. The maim rock units of Gaj formation are sandy limestone with crystalline and pelitic limestone.

The strike and dips in the area show a series of folds and anticlines and synclines are well defined. The dips are generally gentle. According to the previous investigations, drilling along the right abutment (ML-3) shows that core recovery is about 80% and water losses during the water pressure tests are negligible.

Both the abutments show nearly same elevation. The drilling core on the left abutment is similar to that on the right abutment. It is hard as well as compact and few solution cavities are observed. The core recovery is nearly 80% and water losses are negligible (ML-2).

The valley section is about 100 m wide and covered with overburden of different composition, containing loose and calcareously cemented river deposits of sand and gravel with some boulders in thickness of about 5 m. The dip of bed is gentle varying 5°-20°. The dam axis area forms the limb of a syncline. The valley section suggests competent rock foundation with a good core recovery (ML-1).

As far as surface investigation is concerned, the solution cavities are very few, though they are seen on the right abutment only. No major open joints, tension cracks and bigger cavities are observed. Few samples were taken from the recovered core and were tested by WAPDA for determination of the unconfined compressive strength. The results of the laboratory test show that the limestone rock appears to be quite compact and sound enough to sustain the load of proposed structures.

H.3.4 Optimization of Dam Scale

H.3.4.1 Allowable Maximum Reservoir Storage

Study on the required storage capacity of reservoirs was made by WAPDA as explained in Section 2.1. Finally taking into account the flood control by curtailing flood peaks, the active reservoir storage was fixed at 54.7 MCM (crest El. 168.6 m) for the Khadeji dam and 50.9 MCM (crest El. 177.1 m) for the Mol dam, which were fixed at the maximum of the topographic limits (Ref. 01).

After completion of the previous study, about one decade has passed. Several important projects were executed during the decade, which would affect the implementation of this project, especially for the Khadeji dam.

A part of the existing two lanes and other two lanes under construction of the Super Highway from Karachi to the north will be submerged under the Khadeji reservoir, resulting in increase of relocation costs of the Highway. Moreover, the Khadeji dam axis touches the

Precision Engineering Complex (PEC) of Pakistan International Airline (PIA) belonging to the Ministry of Defence, and clearance between the proposed surcharge water level (El. 166.3 m) in the previous study and the lowest ground elevation (El. 169 m) of the Complex is only 2.7 m. Therefore, in order to avoid submergence of the Super Highway, to minimize the costs, as well as to keep more clearance for the PEC, the maximum surcharge elevation would be fixed at El. 163 m for the Khadeji dam.

The maximum allowable surcharge level of the Mol reservoir is El. 175.2 m which shows the allowable topographic limitation as also studied by WAPDA.

(1) Gross Storage Capacity

Based on these conditions and newly prepared elevation-capacity curves discussed in Section H.3.2, the maximum gross storage capacities are fixed at 45.7 MCM for the Khadeji dam and 54.5 MCM for the Mol dam after subtracting flood surcharge depth as discussed in Section H.4.3. The following table shows the summary:

Item	Unit	Khadeji Dam	Mol Dam
1. Maximum Allowable Surcharge Level	El. m	163.0	175.2
2. Surcharge Depth*	m	3.9	3.4
3. Normal Full Water Level	El. m	159.1	171.8

Remarks: Surcharge depth was calculated through the flood routing analysis described in Section H.4.3.

(2) Dead Storage Capacity

The dead storage capacity is defined to be equivalent to the sediment volume to be deposited in the reservoir. As studies in ANNEX-B, based on analysis of suspended load measurement, the design sediment load at the both damsites was estimated at 360 $m^3/km^2/yr$ after allowing for some factors such as river bed load, trap efficiency of 55%, etc., which shows a litter higher value compared to 260 $m^3/km^2/yr$ analysed in the previous study.

Useful life of reservoir is assumed to be 50 years as recommended in the previous study. The dead storage capacities are estimated to be 10.2 MCM for the Khadeji dam and 10.7 MCM for the Mol dam.

Dam	Dead Storage Capacity MCM	Dead Storage Elevation El. m
1. Khadeji Dam	10.2	149.0
2. Mol Dam	10.7	156.5

(3) Maximum Live Storage Capacity

Maximum live storage capacity of the reservoir can be calculated by subtracting dead storage capacity from the maximum gross storage capacity. These were estimated at 35.5MCM for the Khadeji dam and 43.8MCM for the Mol dam.

H.3.4.2 Alternative Study Cases

As discussed in Chapter D.3 of ANNEX-D, groundwater recharge to be augmented by dam(s) has closely related to storage capacity of dams and its combination, dam operation rule and runoff pattern at the damsites. Based on the maximum live storage discussed in Subsection H.3.4.1, seven (7) alternative cases were set for the further water balance study on artificial recharge by dam(s), and their principal features of respective alternatives are presented in Table H.3.2 and summarized below:

Combination of Dam(s)		Khadeji + Mol	Khadeji + Mol	Mol Only	Mol Only	Mol Only	Khadeji Only	Khadeji Only
Description	Case No.	1	2	3	4	5	6	7
1. Live storage capacity	MCM	35.5 +43.8	35.5 +35.0	43.8	35.0	30.0	35.5	30.0
2. Surcharge level	El. m	163.0 and 175.2	163.0 and 173.0	175.2	173.0	171.9	163.0	162.7
3. Dam height	m	40.5 and 51.0	40.5 and 48.8	51.0	48.8	47.7	40.5	40.2

H.3.4.3 Dam Operation Rule

Runoff at the proposed damsite(s) and water demand in the project area are generally decisive factors in determining the reservoir capacity. However, in the project area, there are huge water demands, so limited water resources, and huge groundwater reservoir in the basin. In general, water from a reservoir is released depending on water demand in the downstream area. However, since there is a huge groundwater reservoir with a capacity of more than 300 MCM in the phreatic aquifer, it is not necessary to discharge water from the reservoir according to the water demand.

Moreover, equal recharge to the phreatic aquifer in the upper and lower stretches should be kept as much as possible. If discharge from the dam is small, most water will be recharged to groundwater only in the upper river stretch. On the other hand, in case that the discharge is more than recharge capacity, excess water will be wasted into the sea without being utilized. Therefore, allowable discharge from the dam will only be an important factor for the dam operation.

As discussed in detail in ANNEX-D, allowable discharge from the dam(s) is comprehensively determined from the hydrological and hydrogeological analyses. Finally, it is determined to be 8 m^3/sec (21 MCM/month) at the downstream of the proposed damsites. This is only a dam operation rule given to the proposed dams in this project.

Dam(s) operation rule is set to release water from the dam(s), which corresponds to the balance between the allowance discharge of 8 m^3/sec, and the runoff from the remaining catchment area from the proposed damsites to the National Highway bridge. In case the runoff from the remaining catchment area exceeds the allowable discharge, no water is released from the dam(s). Only balance of water is released, when runoff is less than the allowable discharge.

H.3.4.4 Artificial Recharge by Dams

Natural recharge to the phreatic aquifer in the project basin is estimated at 46.5 MCM/yr on an average for 60 years (1929-1988), as simulated in ANNEX-D. Applying the water balance model installed with the dam operation rule discussed in Subsection H.3.4.3, artificial recharge is calculated for respective alternative cases. The details are described in ANNEX-D, and the following shows the summary of the results:

Unit: MCM

	JICA Study							WAPDA Study
	Case-1	Case-2	Case-3	Case-4	Case-5	Case-6	Case-7	
Combination of Dam	Khadeji + Mol	Khadeji + Mol	Mol Only	Mol Only	Mol Only	Khadeji Only	Khadeji Only	Khadeji +Mol
Live Storage Volume								
Khadeji;	35.5	35.5	43.8	35.0	30.0	35.5	30.0	32.3*
Mol;	+43.8	+35.0	-	-	-	-	-	+33.2*
1. Natural Recharge without Dam	46.5	46.5	46.5	46.5	46.5	46.5	46.5	N.A.
2. Recharge from Precipitation, River Runoff & Dam Spillout	26.1	26.9	39.6	41.1	41.6	43.1	43.8	26.8
3. Contribution to Recharge by Dam(s)	44.5	42.4	25.8	23.6	22.3	19.6	18.3	46.6
4. Total Recharge to Phreatic Aquifer (2+3)	70.6	69.3	65.4	64.7	63.9	62.7	62.1	73.4

Remarks: *; shows only active storage capacity excluding flood control space.
N.A.; No available data in the previous study (Ref. 01).

H.3.4.5 Potential Agricultural Development Area

Total groundwater recharge in the project area is estimated at 62.1-70.6 MCM/yr depending on active reservoir storage and combination of dams, as discussed in Subsection H.3.4.4. Groundwater is mainly extracted for irrigation and domestic water in the project area, and potable water supply to Karachi. Moreover, natural discharge to the sea is inevitable in the lower part of the project area.

Possible irrigation area by using groundwater to be augmented by construction of dam(s) is estimated at 4,100 to 4,860 ha depending on combination of dams and live storage capacity as summarized below:

Unit: MCM

Item	Case-1 Khadeji + Mol	Case-2 Khadeji + Mol	Case-3 Mol Only	Case-4 Mol Only	Case-5 Mol Only	Case-6 Khadeji Only	Case-7 Khadeji Only
Live Reservoir Capacity:							
Khadeji ;	35.5	35.5	-	-	-	35.5	30.0
Mol ;	43.8	35.0	43.8	35.0	30.0	-	-
1. Total recharge to phreatic aquifer	70.6	69.3	65.4	64.7	63.9	62.7	62.1
2. Water supply to Karachi	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Domestic Water Supply	2.3	2.3	2.3	2.3	2.3	2.3	2.3
3. Natural discharge to sea	1.3	1.3	1.3	1.3	1.3	1.3	1.3
4. Available water for irrigation	66.0	64.7	60.8	60.1	59.3	58.1	57.5
5. Net irrigation withdrawal per 1,000 ha*	13.4 - 13.9 (Cropping Intensity = 1.5)						
6. Possible irrigation area (ha)	4,860	4,790	4,420	4,350	4,270	4,160	4,100

Remarks: *; Net irrigation withdrawal = (Irrigation water demand) - (Deep percolation)
Refer to ANNEX-G.

H.3.4.6 Optimization of Dam Scale

(1) Construction Costs

The construction cost comprises the direct construction cost, the operation and economic maintenance cost, administration and engineering costs, and physical contingency. The financial construction costs for respective cases are presented in Table H.3.3 (details in ANNEX-I), and these economic construction costs are summarized below:

Alternative Case	Storage Capacity (MCM) Khadeji	Mol	Economic Construction Cost (10^6 Rs.)	O&M Cost (10^6 Rs.)
Case - 1	35.5	43.8	1,451	8.7
Case - 2	35.5	35.0	1,426	8.7
Case - 3	-	43.8	659	4.3
Case - 4	-	35.0	633	4.3
Case - 5	-	30.0	621	4.3
Case - 6	35.5	-	793	4.3
Case - 7	30.0	-	784	4.3

(2) Project Benefit

Project net incremental benefit to be expected is defined as the difference between with- and without-project conditions. The details are described in ANNEXes-F and J, and the economic net incremental benefit at the full development stage is summarized below:

Alternative Case	Economic Benefit (10^6 Rs.)
Case - 1	104.7
Case - 2	102.5
Case - 3	95.2
Case - 4	93.7
Case - 5	91.3
Case - 6	89.6
Case - 7	88.3

(3) Economic Evaluation

Based on the benefit and economic costs, Benefit Cost ratio (B/C), and Net Present Value (B-C) are calculated on the assumption that the project life is 50 years for evaluation. The results are summarized below:

Case No.	Case-1	Case-2	Case-3	Case-4	Case-5	Case-6	Case-7
	Khadeji + *Mol*	Khadeji + *Mol*	Mol *Only*	Mol *Only*	Mol *Only*	Khadeji *Only*	Khadeji *Only*
Live Reservoir Capacity:							
Khadeji	35.5	35.5	-	-	-	35.5	30.0
Mol	43.8	35.0	43.8	35.0	30.0	-	-
1. Total Ecnomic Construction Cost (Rs. 10^6)	1,451	1,426	659	633	621	793	784
2. Net Incremental Economic Benefit*1 (10^6 Rs.)	104.7	102.5	95.2	93.7	91.3	89.6	88.3
3. EIRR (%)	5.18	5.15	10.40	10.60	10.53	8.40	8.37
4. B/C *1	0.67	0.66	1.33	1.36	1.35	1.05	1.05
5. B-C (Rs. 10^6) *1	-412	-408	188	196	187	36	33

Remarks: Discount rate of 8% is applied.

As seen in the above table,the Khadeji dam with a storage capacity of 35.5 MCM (Case-6) and the Mol dam of 35.0 MCM (Case-4) are the most economical alternatives for a single dam case. These two cases were studied in detail in Chapter H.5 and H.6. The salient features of both dams are given in Table H.3.8 and summary is as follows:

Item	Unit	Khadeji Dam	Mol Dam
1. Dam Type		Concrete gravity	Rockfill
2. Crest Elevation	EL. m	165.0	175.3
3. Normal Full Water Level	EL. m	159.1	169.6
4. Gross Storage Capacity	MCM	45.7	45.7
5. Live Storage Capacity	MCM	35.5	35.0
6. Dam Height	m	40.5	48.8
7. Dam Volume	10^3m^3	159.6	1,730

H.4 FLOOD ROUTING

H.4.1 General

The water level in a reservoir depends on the difference between the discharge rates of inflow and outflow. In ANNEX-B, PMF and respective flood discharges for respective probable floods are calculated, and by construction of dam(s), there is a certain flood storage effect to curtail the peak flood discharge. In this Chapter these analyses for flood mitigation are made for the Khadeji dam wiith a live storage of 35.5 MCM and for the Mol dam of 35 MCM based on the optimization study in Section H.3.4.

H.4.2 Calculation Formulas Adopted

(1) *Spillway Discharge*

The following formula is adopted to determine spillway discharge:

$$Q = C \times L \times H^{3/2} \quad \text{(H.4.1)}$$

where,
- Q : Discharge (m^3/sec)
- C : Variable coefficient of discharge
- L : Effective length of the crest (m)
- H : Total head on the crest, including approach velocity head (m)

(2) Spillway Design Flood

By construction of the dam, the flood peak would be curtailed by storage effect of a reservoir. The water level in a reservoir depends on the difference between discharge rates of inflow and outflow. This is described as the following formula:

$$(I_n + I_{n+1}) \cdot 1/2 \cdot Dt = (Q_n + Q_{n+1}) \cdot 1/2 \cdot Dt + (S_{n+1} - S_n) \quad \text{(H.4.2)}$$

where,
- I_n and I_{n+1} : Inflow discharge at time t_n and t_{n+1}
- Q_n and Q_{n+1} : Outflow discharge at time t_n and t_{n+1}
- S_n and S_{n+1} : Storage volume at time t_n and t_{n+1}
- Dt : $Dt = t_{n+1} - t_n$

H.4.3 Spillway Design Flood

(1) Flood Discharge

As assessed in ANNEX-B, flood discharges for respective probable floods at the Khadeji and Mol damsites are presented in Table H.4.1 and peak discharges for respective return periods are summarized below:

	Unit	Khadeji	Mol
Catchment area	km^2	567	596
Flood discharge			
- PMF	m^3/sec	5,120	4,280
- 1,000 yr.	m^3/sec	3,870	3,240
- 100 yr.	m^3/sec	2,240	1,870
- 50 yr.	m^3/sec	1,820	1,520
- 20 yr.	m^3/sec	1,310	1,090
- 5 yr.	m^3/sec	610	390

(2) Spillway Design Flood Discharge

Based on the formulas (H.4.1 and H.4.2), elevation-storage curves, flood discharge as shown in Table H.4.1, etc., the flood routing through the spillway is carried out to determine the design flood discharge of spillways for respective Khadeji and Mol dams.

Discharges through spillways are calculated as shown in Tables H.4.4 and H.4.8 and summarized below:

Item	Unit	Khadeji Dam	Mol Dam
- Normal full water level (NFWL)	EL. m	159.1	169.6
- Maximum water level (MWL)	EL. m	163.0	173.0
- Flood discharge (PMF)	m^3/sec	5,120	4,280
- Design discharge of spillway	m^3/sec	3,970	4,100
- Curtailing peak discharge	m^3/sec	1,150	180
- Surcharge storage	MCM	37.6	15.3

(3) Design Discharge for Energy Dissipator

Design discharge for energy dissipator of the spillway is determined to be 100 year probable flood. The same procedures for flood routing described above are adopted for its calculation. The results are shown in Tables H.4.5 and H.4.10 and summarized below:

Item	Unit	Khadeji Dam	Mol Dam
- Normal full water level (NFWL)	EL. m	159.1	169.6
- Maximum water level (MWL)	EL. m	159.9	171.6
- Flood discharge (100 yr flood)	m^3/sec	2,240	1,870
- Design discharge of energy dissipator	m^3/sec	1,900	1,770
- Curtailing peak discharge	m^3/sec	340	100

H.4.4 Reservoir Storage Effect for Flood Protection Dikes in the Downstream of the Project Area

Karachi Flood Control Project was implemented by Karachi Development Authority (KDA). The project aims to control flood with a 50-year return period and to protect the urban area located between the National Highway and the sea. The 50-year probable flood adopted by the project is summarized below:

Item	Peak Discharge of *1 50 Year Probable Flood	JICA Estimate
1. Estuary	6,800	-
2. National Highway Bridge	6,380 *2	-
3. Khadeji damsite	2,270	1,820
4. Mol damsite	1,470	1,520
5. Khadeji + Mol	3,740	3,340

Remarks : *1 refer to 01.
*2 calculated on a basis of the river basin ratio

The construction of dam(s) results in decreasing the peak flood due to the storage effect of the reservoir. Peak-cut effect is assessed under the different initial water level. The results are shown in Tables H.4.6 to H.4.8 for the Khadeji dam and Tables H.4.11 to H.4.13 for the Mol dam. Its summary is presented in the following table:

Item		Khadeji Dam			Mol Dam		
Initial Condition							
- Water level	El. m	159.1	155.0	149.0	169.6	163.0	156.5
- Stored volume	MCM	35.5	13.4	0.0	35.0	12.7	0.0
Maximum water level	El. m	159.4	159.3	159.1	171.3	171.2	171.1
Maximum peak discharge*1	m^3/sec	1,820	1,820	1,820	1,520	1,520	1,520
Max. spillage through spillway	m^3/sec	1,610	1,560	1,490	1,430	1,320	1,170
Peak cut	m^3/sec	210	260	330	90	200	350
Spillage starting time*2	hr	0	7	11	0	10	12.5
Peak-cut at the National Highway*3	%	3.3	4.1	5.2	1.4	3.1	5.5

Remarks: *1 50 year probable flood
*2 Difference between starting times of the flood inflow and spillage.
*3 Peak cut ratio against design flood discharge of 6,380 m^3/sec at the National Highway.

As seen in the above table, peak-cut by construction of dam(s) is expected ranging from 1.4% to 5.5% of design flood discharge which will be adopted for the flood control project located at the downstream between the National Highway and the sea.

H.5 DESIGN OF KHADEJI DAM

In this Chapter, the design of Khadeji Dam, which was proposed in the previous study by WAPDA (1982), will be reviewed and revised in consideration of new condition and additional new data obtained through this study.

H.5.1 Damsite and Dam Axis

H.5.1.1 Damsite

Satisfying the following requirements, the damsite was selected at approximately 7 km upper stream from the confluence between Khadeji Nadi and Mol Nadi (See Fig. H.1-1)

- Selection of the location as far downstream as possible from Khadeji Nadi to use the discharge of Khadeji Nadi.
- Selection of the location which is topographically efficient (in terms of the relationship between the dam scale and the reservoir size).
- Selection of the location to be satisfied topographically from the viewpoint of the dam scale.

H.5.1.2 Dam Axis

The following three cases are possible for the dam axis location. Among these Cases, Cases II and III were comparatively studied in "The Feasibility Study Report" by WAPDA in 1982 (See Fig. H.5-1)

- Case I Upstream Plan:
  The location is at about 2 km upstream from the waterfall of Khadeji damsite.
- Case II Midstream Plan: (No. 1 site in the previous study)
  The location is at about 290 m upstream from the waterfall of Khadeji damsite.
- Case III Downstream Plan: (No. 2 site in the previous study)
  The location is at about 1 km downstream from the waterfall of Khadeji damsite.

In this project, Case II was selected because:

- In Case I, the right abutment bed is thin and heavily weathered into the inner area.

- The storage cannot be completely achieved between the location of Case I and II. This factors make the latter advantageous in terms of cost.

- Topographically speaking, deep weathering and high permeability is observed in the left bank of Case II and III, the II and III as well as Case I. However, there is an approximately 5 m wide waterfall, and Case III is less advantageous since its left bank is low and contains solution cavities, compared with Case II. As for the right bank, there is no specific difference between Case II and III.

H.5.2 Type and Construction Method of Dam

H.5.2.1 Type of Dam

In terms of topography, geology, and construction, the concrete gravity dam or the fill dam is suitable here. Especially, from the view point of foundation, the geology is so suitable for both dam types even if the dam height is 40.5 m, because the compressive strength of sandy limestone is around 120 kgf/cm^2.

However, the following condition should be considered.

- The right abutment is hard and shows a very steep slope with a gradient ratio of 1 to 0.65. If the fill dam was selected, excavating would be necessary from the core to anchoring stage.

- Since the values of probable maximum flood and 20 year flood frequency discharge can be as high as 5,120 m^3/sec and 1,310 m^3/sec respectively, fitting of the spillway diversion channel is difficult and the construction cost would considerable increase in the case of the fill dam.

Due to above reasons, the concrete gravity dam is selected the same as the previous study by WAPDA.

H.5.2.2 Construction Method of Dam

With regard to the Khadeji dam, the concrete gravity dam type was selected. There are two kinds of construction methods for concrete gravity dam in general. One is conventional method (columnar block method) and the other is Roller Compacted Concrete (RCC) method.

RCC method has not only advantage but also disadvantage. The advantageous point of RCC method is as follows:

- The construction period of RCC method is roughly one-half to one-third shorter than of a conventional method.

- The construction cost of RCC method is roughly about 60-70% than of a conventional method (See Fig. H.5.2)

However, this method has also the following disadvantageous points:

- The seepage from the dambody,
- The deterioration of the shearing strength of the dambody, and
- The deformation of the foundation

And now, there is no effective way to solve above disadvantage in the case of RCC method. So, the conventional method (columnar block method) should be applied to construct Khadeji Dam.

H.5.3 Dam Crest

(1) Water Level and Storage Capacity

According to studies on the optimization of dam scale and flood routing as described in Subsection H.3.4.6, the optimum scale of dam is decided as follows:

	Water Level (m)	Storage Capacity (MCM)
Dead Water Level (D.W.L)*1	149.0	10.2
Normal Full Water Level (N.F.W.L)*2	159.1	45.7
Maximum Water Level (M.W.L)*3	163.0	83.3

Remarks: *1 See Subsection H.3.4.1.
*2 See Subsection H.3.4.6.
*3 See Section H.4.3.

(2) Dam Crest

In order to determine the dam crest elevation, there are two methods which are based on N.F.W.L. and M.W.L. The dam crest elevation is decided to apply the bigger values resulting from above two calculation methods. (Ref. 02)

The dam crest elevations are given with the conditions of Fetch: 8 km (5 miles) and Wind velocity: 45 m/sec (100 mile/hr) as follows:

Item	Normal Fulll Water Level		Maximum Water Level	
Water Level	El. 159.6	m	El. 163.0	m
Fetch	8	km (5 miles)	8	km (5 miles)
Wind Velocity	45	m/sec (100 mile/hr)	45	m/sec (100 mile/hr)
Freeboard	1.5	m	1.2	m
Dam Crest	El. 161.1	m	El. 164.2	m

The elevation of the bridge for the operation of crest gates is decided as El. 165.0 m in finally taking into account of 0.6 m of the beam height and some allowance. Accordingly, the elevation of dam crest also becomes El. 165.0 m.

H.5.4 Dambody and Dam Foundation

H.5.4.1 Dambody

(1) Standard Cross-section

a. Non-Overflow Section

The basic triangle shape whose crest is EL. 165.0 m is applied for non-overflow section. The downstream incline is 1:0.70, and the upstream one is 1:0.0 (the filet whose incline is 1:0.45 should be settled from El. 133.5 m position), the bases of these inclines are mentioned in the next paragraph. The width of dam crest is 4 m and dam foundation is settled at El. 124.5 m based on Subsection H.5.4.2. In addition, the gallery should be set at El. 130.5 m for the purpose of operation and maintenance of dam. The non-overflow section is illustrated in Fig. H.5-5.

b. Overflow Section

As mentioned in Section H.5.5, the elevation of spillway crest should be settled at El. 155.0 m, and seven (7) radial type gates are recommended as spillway gates, whose dimension are 12.2 m x 4.6 m (40' x 10'). The gallery and the exterior shape of dam body is the same as non-overflow section. The detail of overflow section is illustrated in Fig. H.5-5.

(2) Stability Analysis

a. Earthquake Coefficient

As shown in Fig. H.5.3, Khadeji damsite and Mol damsite are situated in a zone of earthquake coefficient from 1/15 gal to 1/20 gal. Therefore, the earthquake coefficient in this study is applied as 1/10 gal corresponding to the previous study made by WAPDA.

b. Condition of Analysis

<u>Stability condition</u>

- Safety factor of shear friction : $Fs \geq 4$

- Internal friction factor : $f = 0.7$
  (This factor is determined in anticipation of safety to laboratory test data, i.e. $f' = 0.8$, which is expressed ANNEX-C.)

- Shearing strength of bedrock : $t_o = 120$ tf/m^2
  (This value is determined by using laboratory test data, i.e. $t = 150$ tf/m^2 referred in ANNEX-C, and by applying safety rate of 0.8.)

<u>Load condition</u>

Case-1 N.F.W.L 159.1 m	Case-2 M.W.L 163.0 m	Case-3 Empty of reservoir
- Dead weight of dambody - Hydrostatic pressure - Dynamic water pressure - Silt pressure - Uplift - Inertia force by earthquake	(Same as Case-1)	- Dead weight of dambody - Inertia force of dambody by earthquake

c. Items and Formulas of Analysis

- Qualification that the resultant of external force is inside the middle third of bottom length.

$$e = \frac{1}{2} - d < \frac{1}{6}$$

where,
e : Eccentric distance
l : Bottom length = 32.4 (m) (See Fig. H.5-5)
d : Distance from the edge of bottom to the effective point of resultant

- Qualification that the safety factor of shear friction (Fs) is more than four (4).

$$Fs = \frac{to \times l + f \times V}{H} \geq 4$$

where, to : Shearing strength of bedrock = 120 (tf/m^2)

l : Bottom length = 32.4 (m)

f : Internal friction factor = 0.7

V : Vertical force (tf)

H : Horizontal force (tf)

- Qualification that the actual compressive stress is less than the allowable one.

$$s = \frac{V}{l} \times \left(1 \pm \frac{6 \times e}{l}\right) \leq sa$$

where, s : Actual compressive stress

sa : Allowable compressive stress

d. Result of Stability Analysis

The result of stability analysis is shown in following table:

	Case-1	Case-2	Case-3	Remarks
1. Eccentric distance (m)	5.33	5.46*	6.14*	l/6 = 5.4
2. Safety factor of shear friction	4.65	4.59	16.20	
3. Compressive stress (tf/m^2)	76.7 0.5	77.7 -0.4*	97.4 -6.2*	

Remarks : * It is no problem that the resultant is off middle third, because the caused tensile stress is so small.

H.5.4.2 Dam Foundation

(1) Excavation Line

a. Riverbed

Gaj formation is available as the dam foundation, after the 3-10 m elimination of sand and gravel layer of riverbed. The lowest excavation line is settled at El. 124.5 m, after the surface layer consisting of crystalline limestone is eliminated since this surface layer is rather cracky. (See Drilling KD-1)

b. Right Bank

The slope of right bank is steep, and Gaj formation is cropping out here. The surface aspect and the data by electric investigation say that the layer in 10-15 m depth from surface has numerous cracks and has also already weathered. So, the surface layer of right bank should be eliminated about 10-15 m depth in consideration of excavating shape.

c. Left Bank

The topography of left bank is relatively gentle and cracks have not grown remarkablly under the influence of geological structure. However, drilling test suggests that weathering is getting along on. Accordingly the gentle shape of excavation should be recommended including of removal of surface layer about 10-15 m.

On the basis of above discussion, the excavation shape is shown in Fig. H.5-6.

(2) Curtain Grouting

The grouting design is built only upon the purpose of the safety of dambody, since one of objects of dam is to supply the groundwater to aquifer.

a. Grouting Depth

- Riverbed

The lowest line of curtain grouting is determined by the following formula named Simonds formula:

$$d = \frac{h}{3} + C$$

where,
d : Depth (m)
h : Water depth of reservoir
= N.F.W.L. 159.1 - El. 124.5
= 34.6 (m)
C : Constant = 5-24 (m) (Mean = 15)

In accordance with above formula, the lowest line of curtain grouting is settled at El. 94.5 m (depth is 30 m); it means that this line is on the sand stone layer, which has rather stable seepage, in Gaj formation. (See Drilling KD-1)

- Right bank

  The depth of curtain grouting should include greenish pelitic limestone observed at El. 135.0 m.

  For the purpose of constructing facility, wing should have the radial grout (Radius = 30 m) at 10 m right site from the edge of dambody.

- Left bank

  The depth of curtain grouting should include the part of the all seepage and is 30 m. (See Drilling KD-2)

b. Arrangement of Holes

Row spacing is designed for 1 m, and hole spacing for 2 m.

c. Target lugeon value

The target lugeon should be under five (5) in the layer with rather large seepage for the purpose of filling up S.C.

(3) Consolidation Grouting

For the purpose of consolidation at the dam base, the consolidation grouting is applied. The depth of holes is 5 m and intervals are 5 m each row to row and hole to hole.

H.5.5 Spillway

(1) Design Flood Discharge

The design flood discharge and the design discharge for energy dissipater which are determined in Section H.4.3 is shown as below:

Item	Unit	PMF for Spillway	100 Year Flood for Energy Dissipater
Flood discharge	m^3/sec	5,120	2,240
Design discharge	m^3/sec	3,970	1,900

(2) Type

The concrete dam spillway is provided at the overflow section on the dambody in economical reason. Spillway gates are applied on the top of crest for the purpose of increase of the active storage capacity. Based on SectionH.5.5, spillway gates are seven (7) numbers of radial gates having 12.2 m (40 feet) width and 4.5 m (15 feet) height each.

(3) Elevation of Spillway Crest

In order to obtain a large amount of the live storage water, the spillway shall has a large discharge capacity having a large cross sectional area. Therefore, the spillway crest shall be taken down as lower as possible and spillway gates shall be provided as much as wider and taller.

The spillway discharge is calculated by following formula. (Ref. 02)

$$Q = C \cdot L \cdot H^{3/2}$$

where,
- Q : Spillway discharge
- C : Discharge coefficient
- L : Effective length of weir crest
- H : Total head above the crest

In order to determine the elevation of the spillway crest by using above formula, following hydraulic conditions are applied.

$$P/H \geq 0.5$$

where,
- P : Height from the crest to the bottom of approach channel (Sediment EL. 149.9 m)
- H : Total head above the crest

then, H + P = 163.0 (MWL)* - 149.0 (sediment EL.) = 14.0 m

where, P/H is given as 0.75 with some allowance.

therefore, $P = (0.75 \times 14) / (1 + 0.75) = 6.0$ m

Then, the crest elevation = 149.0 + 6.0 = 155.0 m

Remark: * See Section H.4.

(4) Length of Spillway Crest

According to the previous study by WAPDA, five (5) radial gates havings 40' length and 20' width are designed. In this clause the number of gates is reexamined for two cases of five (5) gates and seven (7) gates under following condition.

Conditions :
- M.W.L. 163.0 m (See Section H.4.3)
- EL. of spillway crest 155.0 m (See Subsection H.5.4.3)

When N.F.W.L is 159.1 m (active storage capacity is 35.5 MCM) and the elevation of spillway crest is 155.0 m, M.W.L is estimated as follows:

	Number of Gates	
	5 Nos.	7 Nos.
M.W.L (m)	164.4	163.0

When the both dam scales are the same, in the case of five (5) numbers of gates N.F.W.L must be lower than 159.1 m and active storage capacity must be smaller than 35.5 MCM. Therefore, the case of five (5) gates will be economically less advantageous than the case of seven (7) gates. As the result of above reexamination, the design of seven (7) numbers of gates 12.2 m x 4.6 m (40' x 15') is adopted in consideration of connection with tower rivers.

H.5.6 Outlet Works

Outlet works are comprised of an intake structure and outlet conduit. The maximum intake discharge was fixed to be be 8.0 m^3/sec from the groundwater study in ANNEX-D. However, the maximum intake discharge should be determined under the consideration as following qualification:

- In the case of occurring something wrong in the dambody, it is necessary to make lower the water level of reservoir.

- *If anything wrong should occur after finish ponding, it is necessary to make lower* the water level of reservoir.

For the above reasons, 12 m^3/sec is recommended as the maximum intake discharge, it means a fifty percent raise in 8 m^3/sec of the original maximum discharge. The water taken at the inlet flows down through the pressure pipe to the outlet conduit. The tower type with sluice gates is recommended as intake structure since dam type is concrete gravity dam and it should be settled at the left side of spillway.

The hydraulic energy of water would be dissipated at the downstream of outlet conduit by using the high pressure regulating gate. The diameter of the gate is estimated by the following formula:

$$Q = K \times \sqrt{2gH}$$

$$K = \frac{1}{\sqrt{\Sigma\left(\frac{fi}{ai^2}\right)}}$$

where,	Q	:	Offtake discharge = 12 (m^3/sec)
	H	:	Total head = D.W.L. 149.0 m - EL. 137.1 m = 10.9 (m)
	fi	:	Head loss in each part
	ai	:	Section area of each part
	g	:	Acceleration of gravity

The calculation of water head loss taking of 1.3 m gate diameter and 1.5 m pipe diameter are shown in Table H.5.1. According to Table H.5.1, the offtake discharge is as follows:

$$K = 0.819$$
$$Q = 0.819 \times \sqrt{2g \times 10.9} \div 12.0 \ m^3/sec \geq 12 \ m^3/sec$$

H.5.7 Temporary River Diversion Works

The peak diversion discharge of the design flood is obtained at the 20 year flood and is 1,310 m^3/sec (See Table H.4.1). The river diversion work is considered following 3 cases:

- Case-1

  The first construction work is temporary diversion tunnel by the temporary closure of entire section of stream. And then, the excavation of bed and concreting for dambody will start.

- Case-2

  The first construction work is temporary closure of a half section of stream in order to shift the stream line, and then, the excavation of bed and concreting for dambody will start inside this half closure spot. After finishing this work, the stream line is changed to the diversion channel on the half constructed dambody in order to constructed the rest half dambody.

- Case-3

  While the concrete blocks for the dambody is laid muturally, the flood makes overflow the rather lower blocks.

In the case of Khadeji dam, Case-3 is selected as diversion works in according to the following reasons, as well as the previous study by WAPDA (1982). And in addition, the temporary drain of dambody is recommended for drainage of small scale discharge.

- *The diversion design discharge is so large as 1,310 m^3/sec.* It means that the construction cost of diversion tunnel will get higher on account of the long diameter of tunnel.

- The annual mean rainfall at damsite is around 200 mm, and there are nineteen (19) years during sixty (60) years (1929-1988), which have less than 100 mm rainfall.

- Most of rainfall in a year is recorded during the monsoon season (for three months from July to September).

H.6 DESIGN OF MOL DAM

In this Chapter, the design of Mol Dam, which was proposed in the previous study by WAPDA (1982), will be reviewed and revised in consideration of new condition and additional new data obtained through this study.

H.6.1 Damsite and Dam Axis

(1) Damsite

The location of the damsite is determined at about 9 km upstream from the confluence between Khadeji Nadi and Mol Nadi for the same reasons as for the Khadeji dam (See Fig. H.1-1).

(2) Dam Axis

The ideal location for the dam axis is topographically limited to the position shown in Fig. H.6-1 if the active storage of 35 MCM is taken into account. This dam axis is corresponding with the previous proposal studied by WAPDA, in 1982. If the dam axis was designed to be constructed at the upper stream from this place, it would be impossible to secure the live storage of 35 MCM. On the other hand, if a downstream location was selected, the required volume of the dambody would substantially increase because there is a gentle slope. It, therefore, presents disadvantages in terms of cost.

However, the new dam axis of the left abutment is replaced at about 40 m toward the downstream from the original plan in order to reduce the embankment volume of the dam (See Fig. H.6-1).

H.6.2 Type of Dam

For construction of the Mol dam, a fill type dam was considered in the previous studies by WAPDA. Then, comparative study of two types of fill type and concrete gravity type dams was made, and fill type dam was selected for the following reasons:

- The valley at the Mol damsite is so wide that a width-to-height ratio becomes 10 at EL. 160 m.

- The formation of dam is formed by pelitic limestone and its compressive strength is relatively low, down to 70 to 80 kgf/cm^2. Therefore, there would be difficulties in construction of concrete gravity type dam, if the dam height should be some 50 m.

In the case of fill type dam, the following four types of fill dams can be considered (See Fig. H.6-2).

- Type A : Homogeneous type
- Type B : Zone type
- Type C : Facing type
- Type D : Core type

In the previous study, Type A was selected for the main reason that sufficient fill materials, which are mainly earth, could be secured within a short distance from the damsite and the filter material could also be available.

After making the review of the previous study and of the additional laboratory analysis and information and the results of soil mechanical analysis made by WAPDA, however, Type B, zone type, would be more suitable than Type A mainly for the following reasons:

- According to the previous design, Type A dam would require about 2×10^6 m^3 of fill materials. On the other hand, available volume of fill materials around the proposed damsite to be utilized for core materials would be about 800,000 m^3.

- Sufficient impermeability of dam body could be secured through adequate moisture control of fill materials available around the damsite.

- Sufficient volume of rock materials could be available near the damsite.

- In the case of zone type dam, high stability of the dam body could be expected because of large shearing stress and occurrence of no pour pressure in permeable zone.

- Construction of zone type dam would be easier than Type A dam for which moisture control of earth fill materials over a wide filling area would actually be difficult especially for a big size of homogeneous type of dam.

H.6.3 Dam Crest

(1) Water Level and Storage Capacity

According to studies on the optimumization of dam scale and flood routing as described in Subsection H.3.4.6, optimum scale of dam is decided as follows:

	Water Level (m)	Storage Capacity (MCM)
Dead Water Level (D.W.L)*1	156.5	10.7
Normal Full Water Level (N.F.W.L)*2	169.6	45.7
Maximum Water Level (M.W.L)*3	173.0	61.0

Remarks: *1 See Subsection H.3.4.1.
*2 See Subsection H.3.4.6.
*3 See Section H.4.3.

(2) Dam Crest

In order to determine the dam crest elevation, there are two methods which are based on N.F.W.L and M.W.L. The dam crest elevation is decided to apply the bigger values resulting from above two calculation methods (Ref. 02).

The dam crest elevations are given with conditions of Fetch: 8 km (5 miles) and Wind velocity: 45 m/sec (100 mile/hr) as follows:

Item	Normal Full Water Level		Maximum Water Level	
Water Level	EL. 169.6	m	EL. 173.0	m
Fetch	8	km (5 miles)	8	km (5 miles)
Wind Velocity	45	m/sec (100 mile/hr)	45	m/sec (100 mile/hr)
Freeboard	2.4	m	1.8	m
Protection Zone	0.5	m	0.5	m
Dam Crest	EL. 172.5	m	EL. 175.3	m

Consequently, the dam crest is set at EL. 175.3 m.

H.6.4 Dambody and Dam Foundation

H.6.4.1 Dambody

(1) Standard Cross-section

The following considerations was made for safe and economical construction of the dam, taking into account of the availability of embankment materials near the damsite.

The thickness of the impervious core at Mol dam was taken 50% of the reservoir water depth due to low plasticity of the core material, because of 30 to 50% of the water depth in general cases.

The embankment materials for the dam construction are categorized as follows:

- Crystalline limestone hard rocks obtained at the upstream of left bank can be utilized as an embankment material for Zone 3.
- Fine materiels as a secondary product of rocks for Zone 3 can be utilized for Zone 2, transition zone at the upstream.
- Sandy limestone and pelitic limestone which are moderate and soft rocks are useful for Zone 2 at the downstream.
- Massive hard limestones to take from the quarry site are for the rock facing at the upstream.
- Sand and gravel from the river bed are utilized for filter and drain materials. The filter zone is considered as thicker as possible because the filter materials consist of calcareous sand.

The standard cross-section is shown in Fig. H.6-4, taking into consideration of above categories.

(2) Stability Analysis

a. Earthquake Coefficient

As described in Subsection H.5.4.1, the earthquake coefficient in this study is applied as 1/10 gal.

b. Design Value of Dam Embankment Materials

The design values of dam embankment materials are tabulated as below, based on the results and analysises of the laboratory tests. Details are described in ANNEX-C.

Embankment Material	Wet Density γt (t/m^3)	Saturated Density γsat (t/m^3)	Submerged Density γsub (t/m^3)	Cohesion c' (t/m^2)	Angle of Internal Friction ø' (°)
Impervious Zone (Zone 1)	2.10	2.20	1.20	1.0	30°00'
Transition Zone (Zone 2)	1.65	1.94	0.94	0.0	38°00'
Rock Zone (Zone 3)	1.58	1.90	0.90	0.0	40°00'
Rock Facing	1.58	1.90	0.90	0.0	40°00'
Filter	1.90	2.10	1.10	0.0	30°00'

c. Stability Analysis

The stability of dam against sliding was analyzed by means of sliced slip circle method. A safety factor obtained by the slip circle method is derived by the following formula:

$$Fs = \frac{\Sigma\{c' \cdot l + (N - U - Ne)\tan\phi'\}}{\Sigma(T + Te)}$$

where, Fs : Safety factor

c, c' : Cohesion of material on sliding surface of each slice

- c : For total stress analysis
- c' : For effective stress analysis

ϕ, ϕ' : Angle of share resistance of material on a sliding surface of each slice

- ϕ : For total stress analysis
- ϕ' : For effective stress analysis

l : Length of a sliding surface of each slide

b : Width of each slice

N : Normal load acting on sliding surface of each slice

T : Tangential load acting on sliding surface of each slice

Te : Tangential seismic load acting on sliding surface of each slice

Ne : Normal seismic load acting on sliding surface of each slice

U : Pore pressure acting on sliding surface of each slice

For cohesionless materials, the slip circle method has characteristics that the safety factor would become smaller when the slip circle becomes shallow.

Therefore, the analysis for such case is made by surface plate slicing method as shown below:

$$Fs = \frac{(1 - m.k\frac{\gamma sat}{\gamma'})}{m + \frac{\gamma sat}{\gamma'}.k}\tan\phi'$$

where, Fs : Safety factor

m : Gradient of slope

k : Earthquake coefficient

ϕ' : Angle of internal friction of materials

γsat : Saturated density of material

γ' : Submerged density of material

The above formula was applied to the slope under the water level of reservoir, and for the slope above the water level, the formula can be applied by substituting the wet density (γt) for both γsat and γ'.

d. Results of Stability Analysis

The results of stability analysis are shown in Fig. H.6-6 to Fig. H.6-9 and summarized below:

Method	Case	Water Level of Reservoir		Portion	Earthquake Intensity	Safety Factor
Slip Circle Sliding	1.	M.W.L	173.0	Upstream	0.00	2.081
				Dowonstream	0.00	1.673
	2.	N.F.W.L	169.6	Upstream	0.10	1.247
				Downstream	0.10	1.318
	3.	D.W.L	156.5	Upstream	0.10	1.236
	4.	N.F.W.L ↓ D.W.L	167.6 156.5	Upstream	0.05*	1.210
Surface Plate Sliding				Upstream	0.10	1.257
				Downstream	0.10	1.326

Remarks: * Rapid drawdown of water level is not expected frequently and probability of simultaneous earthquake must be very low. Therefore, a half of earthquake coefficient is adopted.

H.6.4.2 Dam Foundation

(1) Excavation Line

a. Excavation of Core Trench

Riverbed : All riverbed materials, river deposits and terrace deposits are to be removed till Gaj formation. Cracky rocks and sandy limestones are removed as much as possible. The lowest excavation level is EL. 126.5 m according to electric resistivity survey (ME-2).

Right Bank : According to results of drilling (ML-3) and electric resistivity survey (ME-3), the excavation level is 5 m deep from the ground surface. Talus deposit has to remove completely.

Left Bank : As the cracky rocks are assumed to develop at the left bank slope, the rock excavation shall be made 5 to 7 m depth from the ground surface considering of results of the other drilling (ML-2) and electric resistivity survey (ME-1).

b. Foundation Stripping

The surface soil especially organic matter shall be removed. The average removal thickness is assumed as 0.5 m.

(2) Curtain Grouting

The curtain grouting beneath the core zone is applied in order to improve the permeability of the base rock for the prevention of piping through the core zone. This purpose is only for stability of the dambody.

a. Grouting Depth

Riverbed : The grouting depth is 15 m based on 1/3 of the water depth of reservoir considering of the involvement of solved cavities which are located about 20 m deep according to drilling data (ML-1).

Right Bank : There is a possibility to have a connection with soft rocks having solved cavity which are observed on outcrops and pelitic limestones which were confirmed by the drilling data (ML-3) at 13 to 19 m deep. Therefore, the grouting depth is scheduled as 10 to 15 m deep involving above rocks.

Left Bank : At a cliff part 15 m depth is applied same as the riverbed. At other parts are 10 m deep according to results of drilling (ML-2) and electric resistivity (ME-1).

b. Arrangement of Holes

Grouting holes are arranged 2 m intervals each row to row and hole to hole.

c. Target Lugeon Value

Taking into considerations of the piping prevention in the core zone and soft bed rocks Lugeon Value shall be less than 5.

(3) Consolidation Grouting

For the purpose of the piping prevention at the core zone foundation, the consolidation grouting is applied at the entire core base. The depth of holes is 5 m and intervals are 3 m each row to row and hole to hole.

H.6.5 Spillway

(1) Design Flood Discharge

The design flood discharge and the design discharge for energy dissipater which are determined in Section H.4.3 is shown as below:

Item	Unit	PMF for Spillway	100 Year Flood for Energy Dissipater
Flood discharge	m^3/sec	4,280	1,870
Design discharge	m^3/sec	4,100	1,770

(2) Location of Spillway

The spillway shall be provided on the reliable ground in geotechnically so as to a fill type dam. The location of the spillway is planned at LSTANO.6 - LSTANO.14 in the left bank with reasons of:

- A wide spillway is required due to large amount of P.M.F for dam safety as 4,280 m^3/sec.
- The spilled flood after dissipated energy shall be discharged to the down stream river without trouble.
- The rock condition of the left bank is better than the right bank and excavated rocks are utilized as an embankment material.

(3) Type of Spillway

The type of the spillway was selected an overflow type with straight crest and without gate. This type of spillway has advantages of a safety of flood discharge and an easiness for the operation and maintenance.

(4) Length of Spillway Crest

According to the previous study by WAPDA, the length of the spillway crest is designed at 244 m (800 feet). In this clause it is reexamined for two (2) cases of 244 m and 320 m which is maximum length in topographically.

When N.F.W.L is 169.6 m (active storage capacity is 35.0 MCM), M.W.L and the dam crest are estimated as follows:

	Length of Spillway Crest	
	244 m (800 feet)	320 m (1,050 feet)
M.W.L (m)	173.6	173.0
Dam Crest (m)	175.9	175.3

The costs of dambody (filling) and spillway (concrete) are roughly estimated respectively as follows:

		Length of Spillway Crest	
		244 m (800 feet)	320 m (1,050 feet)
Dambody	(x 10^3 Rs.)	220,200	211,700
Spillway	(x 10^3 Rs.)	20,100	26,400
Total	(x 10^3 Rs.)	240,300	238,100

As the result of above reexamination, the length of spillway crest should be designed at 320 m (1,050 feet) which has economical advantage.

H.6.6 Outlet Works

Outlet works are comprised of an intake structure and an outlet conduit. The maximum intake discharge was fixed to be 8.0 m^3/sec from the groundwater study in ANNEX-D.

The intake has a functioning to divert the water discharge of 40.5 m^3/sec at the intake water level of EL. 163.1 m which is in-between N.F.W.L EL. 169.6 m and D.W.L EL. 156.5 m, taking into consideration of following conditions:

The drawdown discharge of the reservoir water in emergency, it is necessary to release the active storage of 35 MCM within 10 days and an average discharge is calculated to be 40.5 m^3/sec.

The diverted water is passed through the pressured pipe from the inlet to the outlet conduit. The outlet works is provided into the dambody at the left abutment as a bottom outlet type structure and the intake structure is applied a drop inlet type.

The hydraulic energy of water would be dissipated at the downstream of outlet conduit by using the high pressure regulating gate. The diameter of the gate is estimated by the following formula:

$$Q = K \times \sqrt{2gH}$$

$$K = \frac{1}{\sqrt{\Sigma \left(\frac{fi}{ai^2}\right)}}$$

where,

Q	:	Offtake discharge = 40.5 (m^3/sec)
H	:	Total head = EL. 163.1 m - EL. 135.5 m = 27.6 (m)
fi	:	Head loss in each part
ai	:	Section area of each part
g	:	Acceleration of gravity

The calculation of water head loss taking of 2.1 m gate diameter and 2.4 m pipe diameter are shown in Table H.6.1.

According to Table H.6.1, the offtake discharge is as follows:

$$K = 1{,}751$$

$$Q = 1{,}751 \times \sqrt{2g \times 27.6} = 40.7 \text{ m}^3\text{/sec} > 40.5 \text{ m}^3\text{/sec}$$

H.6.7 Temporary River Diversion Works

The peak diversion discharge of the design flood is obtained at the 20 year flood and is 1,090 m^3/sec (See Table H.4.1).

The river diversion work is considered following 2 cases:

Case-1 : A temporary coffer dam acrossing the entire river and a temporary diversion tunnel through a river bank are provided, and the runoff river water is diverted into the diversion tunnel during the period of the dam construction including excavation, grouting and embankment.

Case-2 : A temporary coffer dam surrounding a half section of the river is provided at the first stage to carry out a half part of the dam construction works and the diversion channel construction into the dam, within the surrounded area by the coffer dam. At the second stage, the runoff river water is diverted into the diversion channel with the other half side of coffer dam to carry out the dam construction works for a remaining part. The diversion channel is utilized as a permanent outlet structure usually.

In case of Mol Dam, "Case-2" is applicable due to following reasons, and the second stage river diversion and the dam embankment can be carried out in the final year.

- In case of providing a left bank diversion tunnel, it is necessary to construct the tunnel of 280 m long and 13 m diameter. The covering thickness of the tunnel is rather small and the reinforcement may be required. So, it is uneconomically.

- In case of providing a right bank diversion tunnel, it is necessary to construct the tunnel of 390 m long and 13 m diameter. The covering thickness of the tunnel at the down stream part from the dam axis is very small and the reinforcement may be required. Moreover, transition channels at the inlet and the outlet of the tunnel shall be provided. So that it is rather costly.

- The bottom outlet at the left bank abutment is required 190 m long and 13 m diameter with 37,000 m^3 concrete volume. It takes long term construction period and considerable construction costs.

In case of a half river coffering as shown in Fig. H.6-10, the section area of the flood way for 1,090 m^3/sec discharge is required 30 m wide and 7.4 m deep. Then, the minor river protection to discharge the flood in safety is to provided 8 m high in the river. The embankment of 550,000 m^3 for this 30 m wide section is possible to undertake within one year.

H.7 FUTURE INVESTIGATIONS FOR MOL DAM

The preliminary design of the Mol dam is carried out totally based on the topo-graphic map (1/2,400) prepared by WAPDA in the previous study (Ref. 01), and geological data and information in the report, and further additional surface geological investigations during this study. Therefore, it is recommended to carry out further topographical and geological surveys necessary for the detailed design.

The following investigations and surveys will be essential for the detailed design for the Mol dam:

1) Topographic Survey

- Confirmation survey of topographic map with a scale of 1/2,400 prepared by WAPDA,
- Profile and cross section surveys along the proposed dam axis,
- Topographic map with a scale of 1/1,000 covering the intake and outlet structures, and
- Mesh survey in the upstream and downstream of the proposed spillway.

2) Core drilling

- Core drilling and water pressure test along the dam axis and the up- and downstream of the main dambody (estimated total length is about 450 m)
- Core drilling at the proposed quarry sites located in the upstream of the spillway and/or the quarry proposed at Khadeji dam, and spillway excavation area (total length is about 150 m).

3) Filling Material Survey

- Test pitting in the proposed borrow area located at about 2-3 km downstream of the Mol dam (test pitting Nos.: about 15).

4) Laboratory Tests

a. Rock Test

- Rock samples from the damsite and quarry site(s) will be tested for the following items: Specific gravity, Absorption, Unconfined Strength, Triaxial Compression.

b. Soil Test

- Soil samples form the damsite and quarrysite(s) will be tasted for the following items: Specific gravity, Absorption, Consistency, Grain size analysis, Compaction, Permeability, and Triaxial Compression (CU, UU).

LIST OF REFERENCES

01 WATER RESOURCES DEVELOPMENT IN MALIR BASIN, FEASIBILITY STUDY, WAPDA, 1979 (UPDATED IN 1982)

02 KARACHI FLOOD CONTROL PLAN, FEASIBILITY REPORT, WAPDA, 1985

03 DESIGN OF SMALL DAM, USBR

TABLES

Table H.2.1 SALIENT FEATURES OF KHADEJI AND MOL DAMS PROPOSED BY WAPDA

		Unit	Khadeji Dam	Mol Dam
a)	General			
	Location		7.2 km upstream of Super Highway Bridge at a distance of about 50 km from Karachi.	8.3 km upstream of Super Highway Bridge.
	River		Khadeji tributary of Maril River	Mol tributary of Maril River
	Type of dam		Concrete Gravity	Earthfill (Homogenious)
	Purpose		Groundwater recharge (Irrigation +Flood +Drinking Water Supply)	Groundwater recharge (Irrigation +Flood+Drinking Water Supply)
b)	Hydrology			
	Catchment area	km^2	567	611
	Mean annual rainfall	mm	217	217
	Minimum annual	MCM	31.2	33.7
c)	Reservoir			
	Livestorage	MCM	32.3	33.2
	Flood control storage	MCM	22.4	17.8
	Dead storage	MCM	7.2	7.7
	Gross storage	MCM	61.9	58.7
	Maximum reservoir area	km^2	14.2	6.3
d)	Dam			
	Type		Concrete Gravity	Earthfill (Homogenious)
	Maximum height	m	39.0	44.2
	Length of crest	m	381	2,347
	Top width	m	9.1	12.2
	Top elevation of dam	EL. m	168.6	177.1
	Nomal full water level	EL. m	162.6	170.7
	Maximum water level	EL. m	166.3	174.7
	Slope: Upstream		1 : 0.1	1 : 3.0
	Downstream		1 : 0.7	1 : 2.0
e)	Spillway			
	Type		Overflow (gated)	Submerged weir (ungated)
	Gates	No. x m x m	5 x 12.2 x 6.1	-
	Capacity	m^3/sec	3,830	3,720
	Reservoir absorption	MCM	48.6	23.2
	Surcharge for design flood	m	3.7	4.0
	Crest elevation	EL. m	156.5	170.7
	Energy dissipation		Stilling Basin (Energy dissipation by hydraulic jump)	No Stilling Basin
f)	Off-take Structure		1.8 m dia circular conduit with 1.8 m x 1.8 m control gate at inlet end through middle of overflow section discharge directly into main stilling basin.	1.8 m dia tunnel with control gate at outlet end and emergency control gate at inlet end with stilling basin on downstream end.
	Outfall Channel		-	6.5 ft. wide channel with 1-1/2 : 1 side slope discharging into natural Nullah.
g)	Irrigation System		Recharging aquifer by controlled releases from Khadeji & Mol Dams	
h)	Cropping Area	ha	5,670	
	Drinking Water Supply	MCM	13.4	

Source : Ref. 01

Table H.3.1 WATER BALANCE OF DAM(S) AND RECHARGE TO AQUIFER

Unit: MCM/year

Item	Case-1 Khadeji +Mol	Case-2 Khadeji +Mol	Case-3 Mol Only	Case-4 Mol Only	Case-5 Mol Only	Case-6 Khadeji Only	Case-7 Khadeji Only
Live Reservoir Capacity							
Khadeji :	35.5	35.5	-	-	-	35.5	30.0
Mol :	43.8	35.0	43.8	35.0	30.0	-	-
Water Balance of Dam(s)							
(1) Present Run-off at Damsite							
Khadeji :	33.9	33.9	33.9	33.9	33.9	33.9	33.9
Mol :	44.8	44.8	44.8	44.8	44.8	44.8	44.8
(2) Controlled Outflow from Dam							
Khadeji :	19.3	19.3	-	-	-	19.7	18.4
Mol :	25.2	23.2	25.8	23.7	22.4	-	-
(3) Uncontrolled Outflow from Dam							
Khadeji :	13.2	13.2	-	-	-	13.1	14.6
Mol :	18.5	20.7	18.3	20.5	21.9	-	-
(4) Total Outflow (2) + (3)							
Khadeji :	32.5	32.5	-	-	-	32.8	33.0
Mol :	43.7	43.9	44.1	44.2	44.3	-	-
(5) Total Loss Volume from Reservoir							
Khadeji :	1.4	1.4	-	-	-	1.1	0.9
Mol :	1.1	0.9	0.7	0.6	0.5	-	-
Water Balance of Recharge to Acquifer							
(6) Present Run-off at National Highway Bridge	93.2	93.2	93.2	93.2	93.2	93.2	93.2
(7) Controlled Run-off at National Highway Bridge	90.5	90.7	92.4	92.5	92.6	92.0	92.2
(8) Natural Recharge without Dam(s)	46.5	46.5	46.5	46.5	46.5	46.5	46.5
(9) Recharge from Precipitation, River Run-off & Dam Spillout	26.1	26.9	39.6	41.1	41.6	43.1	43.8
(10) Contribution to Recharge by Dam(s)	44.5	42.4	25.8	23.6	22.3	19.6	18.3
(11) Total Recharge to Phreatic Aquifer (9) + (10)	70.6	69.3	65.4	64.7	63.9	62.7	62.1

Remarks : See ANNEX-D

Table H.3.2 SALIENT FEATURES OF ALTERNATIVE DAMS

Khadeji Dam	Unit	Live Storage 35.5 MCM	Live Storage 30MCM	WAPDA 54.7 MCM
1. Dead Storage Capacity	MCM	10.2	10.2	7.2
2. Gross Storage Capacity	MCM	45.7	40.2	61.9
3. D. W. L.	EL. m	149.0	149.0	154.3
4. N. F. W. L.	EL. m	159.1	158.5	162.6
5. EL. of Spillway Crest	EL. m	155.0	154.7	156.5
6. Gate of Spillway (W=40')	Nos.	7	7	5
7. M. W. L.	EL. m	163.0	162.7	166.3
8. Freeboard for M. W. L.	m	2.0	2.0	2.3
9. Dam Crest	EL. m	165.0	164.7	168.6
10. Excavation Line	EL. m	124.5	124.5	124.0
11. Dam Height	m	40.5	40.2	44.6
12. Volume of Dambody	1,000 m*3	159.6	157.3	170.1
13. Rough Construction Cost	10*6 Rs.	554.5	547.8	346.1 *

Mol Dam	Unit	Live Storage 43.8 MCM	Live Storage 35MCM	Live Storage 30 MCM	WAPDA 50.9 MCM
1. Dead Storage Capacity	MCM	10.7	10.7	10.7	7.7
2. Gross Storage Capacity	MCM	54.5	45.7	40.7	58.7
3. D. W. L.	EL. m	156.5	156.5	156.5	156.8
4. N. F. W. L.	EL. m	171.8	169.6	168.5	170.7
5. EL. of Spillway Crest	EL. m	171.8	169.6	168.5	170.7
6. M.W.L.	EL. m	175.2	173.0	171.9	174.8
7. Freeboard for M. W. L.	m	1.8	1.8	1.8	2.3
8. Protection Zone	m	0.5	0.5	0.5	-
9. Dam Crest	EL. m	177.5	175.3	174.2	177.1
10. Excavation Line	EL. m	126.5	126.5	126.5	128.5
11. Dam Height	m	51.0	46.8	47.7	48.6
12. Volume of Dambody	1,000 m*3	1,980	1,730	1,606	2,695
13. Rough Construction Cost	10*6 Rs.	459.4	439.5	431.0	242.6 *

Remarks :
D. W. L. : Dead Water Level
N. F. W. L. : Normal Full Water Level
M. W. L. : Maximum Water Level
* : 1980 Price Level

Table H.3.3 PRELIMINARY COST ESTIMATE

Unit: million Rs.

Item	Case-1 Khadeji +Mol	Case-2 Khadeji +Mol	Case-3 Mol Only	Case-4 Mol Only	Case-5 Mol Only	Case-6 Khadeji Only	Case-7 Khadeji Only
Live Storage Capacity (MCM)							
Khadeji :	35.5	35.5	-	-	-	35.5	30.0
Mol :	43.8	35.0	43.8	35.0	30.0	-	-
1. Preparatory Works	92.8	91.5	34.3	32.9	32.3	58.5	57.9
2. Dam							
Excavation	126.7	132.8	73.6	79.7	84.2	53.1	52.4
Filling	239.4	216.3	239.4	216.3	204.8	-	-
Foundation treatment	30.2	30.2	15.7	15.7	15.7	14.5	14.5
Concrete works	411.4	408.5	75.3	72.4	70.9	336.1	331.4
Steel pipe	3.5	3.5	2.9	2.9	2.9	0.6	0.3
Outlet gates	42.4	42.4	25.4	25.4	25.4	17.0	17.0
Spillway gates	39.9	39.9	-	-	-	39.9	39.0
Diversion works	13.5	13.5	13.5	13.5	13.5	-	-
3. Land Acquisition	98.9	98.9	12.5	12.5	12.5	86.4	86.4
4. Access Road	7.9	7.9	1.1	1.1	1.1	6.8	6.8
Total (2 - 4)	1,013.8	993.9	459.4	439.5	431.0	554.5	547.8
5. Causeway	6.2	6.2	6.2	6.2	6.2	6.2	6.2
6. P. Demonstration Farm	13.3	13.3	13.3	13.3	13.3	13.3	13.3
7. Project Office	1.8	1.8	1.2	1.2	1.2	1.2	1.2
8. O&M Equipment	15.5	15.5	10.3	10.3	10.3	10.3	10.3
Total (2 - 8)	1,050.6	1,032.7	490.3	470.5	462.0	585.4	578.8
9. *Physical Contingency*	*161.9*	158.5	73.5	70.9	69.4	88.0	86.3
Total (1 - 9)	1,305.3	1,280.7	598.1	574.3	563.7	732.0	723.0
10. Administration Cost	11.0	11.0	6.7	6.7	6.7	8.0	8.0
11. Engineering Services	172.7	170.3	79.2	76.0	74.6	94.0	93.0
Grand Total	1,489.0	1,462.0	684.0	657.0	645.0	834.0	824.0

Remarks : See ANNEX-I

Table H.3.4 CALCULATION OF STORAGE VOLUME AT KHADEJI DAM

Elevation m	Area 1,000 m*2	Ave. Area 1,000 m*2	Height m	Volume 1,000 m*3	Total Volume 1,000 m*3
(EL. 420 ft.) EL. 128.0	0				
(EL. 460 ft.) EL. 140.2	382	191	12.2	2,331	2,331
(EL. 480 ft.) EL. 146.3	843	613	6.1	3,737	6,067
(EL. 500 ft.) EL. 152.4	2,119	1,481	6.1	9,035	15,102
(EL. 520 ft.) EL. 158.5	6,477	4,298	6.1	26,219	41,321
(EL. 540 ft.) EL.164.6	14,354	10,366	6.1	63,231	104,552
(EL. 560 ft.) EL.170.7	23,230	18,742	6.1	114,328	218,881

Table H.3.5 CALCULATION OF STORAGE VOLUME AT MOL DAM

Elevation m	Area 1,000 m*2	Ave. Area 1,000 m*2	Height m	Volume 1,000 m*3	Total Volume 1,000 m*3
(EL. 450 ft.) EL. 137.2	0				
(EL. 470 ft.) EL. 143.3	303	151	6.1	924	924
(EL. 490 ft.) EL. 149.4	596	449	6.1	2,740	3,664
(EL. 510 ft.) EL. 155.4	1,232	914	6.0	5,482	9,146
(EL. 530 ft.) EL. 161.5	2,295	1,764	6.1	10,757	19,904
(EL. 550 ft.) EL. 167.6	3,565	2,930	6.1	17,872	37,776
(EL. 570 ft.) EL. 173.7	5,212	4,388	6.1	26,768	64,544
(EL. 580 ft.) EL. 176.8	5,958	5,585	3.1	17,313	81,857

Table H.3.6 COMPARISON OF SALIENT FEATURE AT KHADEJI DAM

		By WAPDA	By JICA
a)	Hydrology		
	Catchment area	567 km^2 (219 $miles^2$)	567 km^2 (219 $miles^2$)
	Rainfall		
	Mean annual rainfall	216.7 mm	214.8 mm
	River discharge		
	Mean annual	31.2 MCM	33.9 MCM
b)	Reservoir		
	Live storage	54.7 MCM	35.5 MCM
	Dead storage	7.2 MCM	10.2 MCM
	Gross storage	61.9 MCM	45.7 MCM
c)	Dam		
	Type	Concrete Gravity	Concrete Gravity
	Maximum height	39.0 m	40.5 m
	Length of crest	381 m	310 m
	Top width	9.1 m	4.0 m
	Top elevation of dam	168.6 m	165.0 m
	Normal full water level	162.6 m	159.1 m
	Maximum reservoir level	166.3 m (545.75 feet)	163.0 m (535 feet)
	Slope		
	Upstream	1 : 0.1	1 : 0 (with fillet)
	Downstream	1 : 0.7	1 : 0.70
d)	Spill Way		
	Type	Overflow (Gated)	Overflow (Gated)
	Gates	5 Nos. 12.2 m x 6.1 m (40' x 20')	7 Nos. 12.2 m x 4.6 m (40' x 15')
	Capacity	3,830 m^3/sec	3,970 m^3/sec
	Probable maximum flood	5,210 m^3/sec (*1)	5,120 m^3/sec
	Crest elevation	156.5 m	155.0 m

Remarks : (*1) : Maximum peak discharge of 1,000 year flood

Table H.3.7 COMPARISON OF SALIENT FEATURE AT MOL DAM

	By WAPDA	By JICA
a) Hydrology		
Catchment area	611 km^2 (236 $miles^2$)	596 km^2 (230 $miles^2$)
Rainfall		
Mean annual rainfall	216.7 mm	230.7 mm
River discharge		
Mean annual	33.7 MCM	44.9 MCM
b) Reservoir		
Live storage	50.9 MCM	35.0 MCM
Dead storage	7.7 MCM	10.7 MCM
Gross storage	58.7 MCM	45.7 MCM
c) Dam		
Type	Earthfill (Homogeneous)	Rockfill (Zone type)
Maximum height	44.2 m	48.8 m
Length of crest	2,347 m	2,347 m
Top elevation of dam	177.1 m	175.3 m
Normal full water level	170.7 m	169.6 m
Maximum reservoir level	174.8 m	173.0 m
Slope		
Upstream	1 : 3.0	1 : 2.5
Downstream	1 : 2.0	1 : 2.0
d) Spill Way		
Type	Submerged weir (Ungated)	Submerged weir (Ungated)
Design Capacity	3,720 m^3/sec	4,100 m^3/sec
Probable maximum flood	3,910 m^3/sec (*1)	4,280 m^3/secc
Crest elevation	170.7 m	169.6 m

Remarks : (*1) : Maximum peak discharge of 1,000 year flood

Table H.3.8 SALIENT FEATURES OF KHADEJI AND MOL DAMS PROPOSED BY JICA

Item	Khadeji Dam	Mol Dam
a) Hydrology		
Catchment area	567 km^2 (219 $miles^2$)	596 km^2 (230 $miles^2$)
Rainfall mean annual rainfall	214.8 mm	230.7 mm
River discharge mean annual	33.9 MCM	44.9 MCM
b) Reservoir		
Live storage	35.5 MCM (28,780 A.F)	35.0 MCM (28,380 A.F)
Dead storage	10.2 MCM (8,270 A.F)	10.7 MCM (8,670 A.F)
Gross storage	45.7 MCM (37,050 A.F)	45.7 MCM (37,050 A.F)
c) Dam		
Type	Concrete Gravity	Rockfill (Zone type)
Maximum height	40.5 m (133 feet)	48.8 m (160 feet)
Length of crest	310 m (1,017 feet)	2,347 m (7,700 feet)
Top width	4.0 m	10 m
Top elevation of dam	165.0 m (541 feet)	175.3 m (575 feet)
Normal full water level	159.1 m (522 feet)	169.6 m (556 feet)
Maximum reservoir level	163.0 m (535 feet)	173.0 m (568 feet)
Slope		
Upstream	1:0 (with fillet)	1:2.5
Downstream	1:0.70	1:2.0
d) Spillway		
Type	Overflow (Gated)	Submerged weir (ungated)
Gates	7 Nos. 12.2 m x 4.6 m (40' x 15')	-
Capacity	3,970 m^3/sec (140,180 cusec)	4,100 m^3/sec (144,770 cusec)
Probable maximum flood	5,120 m^3/sec (180,790 cusec)	4,280 m^3/sec (151,130 cusec)
Crest elevation	155.0 m (509 feet)	169.6 m (556 feet)

Table H.4.1 PROBABLE MAXIMUM FLOOD (1/2)

Khadeji Dam Site (567Km2) Unit: m3/sec

Duration	Return Period (Year)							
(hr)	PMF	1,000	100	50	20	10	5	2
1.0	16	12	7	6	4	3	2	0
2.0	156	118	68	55	40	29	18	5
3.0	814	615	355	289	207	149	95	24
4.0	2,759	2,085	1,205	979	701	507	322	82
5.0	4,012	3,033	1,756	1,426	1,023	741	473	124
6.0	4,945	3,739	2,166	1,760	1,262	915	585	155
7.0	5,117	3,869	2,240	1,820	1,306	946	605	160
8.0	4,858	3,672	2,123	1,723	1,235	893	569	145
9.0	4,565	3,446	1,987	1,610	1,150	828	522	123
10.0	4,453	3,357	1,928	1,558	1,108	792	493	104
11.0	4,150	3,123	1,784	1,437	1,016	720	440	81
12.0	3,914	2,939	1,669	1,340	940	659	393	63
13.0	3,678	2,756	1,555	1,243	865	599	348	50
14.0	3,477	2,599	1,456	1,159	800	547	308	41
15.0	3,250	2,424	1,347	1,068	730	492	266	33
16.0	3,072	2,285	1,260	994	673	446	232	27
17.0	2,906	2,156	1,180	927	621	405	200	23
18.0	2,738	2,027	1,099	859	568	363	169	19
19.0	2,586	1,909	1,025	796	520	325	141	16
20.0	2,413	1,774	942	726	466	283	115	13
21.0	2,218	1,623	848	648	407	240	93	11
22.0	1,988	1,446	738	557	342	199	77	9
23.0	1,754	1,265	626	464	281	163	64	8
24.0	1,523	1,086	516	373	224	131	53	7
25.0	1,330	937	424	299	180	106	44	5
26.0	1,177	818	351	242	147	88	36	4
27.0	1,055	724	292	200	123	73	31	4
28.0	948	641	242	165	102	61	26	3
29.0	857	571	200	137	85	51	21	3
30.0	778	510	166	115	71	42	18	2
31.0	772	504	165	118	75	47	22	4
32.0	663	421	122	86	54	33	15	2
33.0	609	379	103	73	46	28	13	2
34.0	560	341	87	62	39	24	11	2
35.0	512	304	73	52	32	20	9	1
36.0	466	268	60	43	27	16	7	1
37.0	417	231	48	34	21	12	5	0
38.0	374	198	39	27	17	10	4	0
39.0	328	167	32	22	14	8	3	0
40.0	276	137	26	18	11	6	2	0
41.0	225	111	21	15	9	5	2	0
42.0	180	90	17	12	7	4	1	0
43.0	144	73	14	10	6	3	1	0
44.0	118	60	12	8	5	3	1	0
45.0	97	50	10	6	4	2	0	0
46.0	80	41	8	5	3	1	0	0
47.0	66	34	6	4	2	1	0	0
48.0	55	28	5	3	2	1	0	0
49.0	43	21	3	2	1	0	0	0
50.0	35	17	2	1	1	0	0	0
51.0	29	14	1	1	0	0	0	0
52.0	24	11	1	0	0	0	0	0
53.0	19	9	1	0	0	0	0	0
54.0	16	7	0	0	0	0	0	0
55.0	13	6	0	0	0	0	0	0
56.0	11	5	0	0	0	0	0	0
57.0	9	4	0	0	0	0	0	0
58.0	7	3	0	0	0	0	0	0
59.0	6	3	0	0	0	0	0	0
60.0	5	2	0	0	0	0	0	0
61.0	4	2	0	0	0	0	0	0
62.0	3	1	0	0	0	0	0	0
63.0	3	1	0	0	0	0	0	0
64.0	2	1	0	0	0	0	0	0
65.0	2	1	0	0	0	0	0	0
66.0	1	0	0	0	0	0	0	0
67.0	1	0	0	0	0	0	0	0
68.0	0	0	0	0	0	0	0	0
69.0	0	0	0	0	0	0	0	0
70.0	0	0	0	0	0	0	0	0
71.0	0	0	0	0	0	0	0	0
72.0								

Table H.4.1 PROBABLE MAXIMUM FLOOD (2/2)

Mol Dam Site (596Km2) Unit: m3/sec

Duration (hr)	PMF	1,000	100	50	20	10	5	2
	Return Period (Year)							
1.0	8	6	4	3	2	2	1	0
2.0	41	31	18	14	10	8	5	1
3.0	119	90	52	43	30	22	13	4
4.0	467	353	205	166	119	86	52	14
5.0	1,419	1,071	621	505	361	263	157	42
6.0	2,256	1,704	989	804	576	419	238	69
7.0	2,893	2,188	1,269	1,032	740	537	289	89
8.0	3,600	2,724	1,579	1,283	920	667	348	110
9.0	4,085	3,092	1,790	1,453	1,041	754	380	122
10.0	4,280	3,240	1,872	1,518	1,087	784	379	122
11.0	4,036	3,055	1,760	1,425	1,017	731	334	107
12.0	4,096	3,096	1,778	1,437	1,022	730	333	98
13.0	3,971	2,998	1,714	1,382	978	695	313	85
14.0	3,818	2,878	1,638	1,317	927	653	292	72
15.0	3,685	2,773	1,570	1,259	880	614	275	61
16.0	3,484	2,616	1,472	1,177	816	564	249	51
17.0	3,319	2,487	1,390	1,107	762	520	227	43
18.0	3,182	2,380	1,322	1,049	716	483	208	37
19.0	3,053	2,279	1,257	993	672	447	189	32
20.0	2,904	2,162	1,184	931	624	408	166	28
21.0	2,775	2,061	1,120	876	581	374	148	25
22.0	2,610	1,933	1,039	808	530	334	128	22
23.0	2,432	1,795	953	735	476	297	111	19
24.0	2,244	1,648	862	658	422	261	96	17
25.0	2,063	1,507	774	584	373	229	84	15
26.0	1,877	1,362	684	510	324	199	74	14
27.0	1,696	1,222	597	440	279	172	64	12
28.0	1,546	1,105	524	383	244	152	58	11
29.0	1,414	1,002	461	335	214	134	51	10
30.0	1,292	908	403	292	187	117	45	8
31.0	1,181	821	351	255	163	103	39	7
32.0	1,086	747	308	224	144	91	35	7
33.0	1,002	682	271	198	128	81	31	6
34.0	926	623	239	176	113	72	28	5
35.0	860	571	213	157	102	64	25	5
36.0	791	518	187	138	89	56	22	4
37.0	683	435	145	106	67	41	14	2
38.0	618	385	123	89	56	34	11	2
39.0	556	337	103	74	46	28	9	1
40.0	495	292	87	62	38	23	7	0
41.0	436	252	72	51	31	18	6	0
42.0	378	215	60	42	26	14	5	0
43.0	325	182	50	35	21	11	3	0
44.0	276	153	41	28	17	9	3	0
45.0	233	129	34	23	13	7	2	0
46.0	196	109	27	18	10	5	1	0
47.0	166	91	22	14	8	4	1	0
48.0	140	76	17	11	6	3	0	0
49.0	118	64	13	8	5	2	0	0
50.0	99	53	10	6	3	1	0	0
51.0	83	43	7	4	2	1	0	0
52.0	69	35	5	2	1	0	0	0
53.0	58	29	3	1	0	0	0	0
54.0	48	23	1	0	0	0	0	0
55.0	41	19	1	0	0	0	0	0
56.0	35	16	1	0	0	0	0	0
57.0	29	14	0	0	0	0	0	0
58.0	25	11	0	0	0	0	0	0
59.0	21	9	0	0	0	0	0	0
60.0	17	8	0	0	0	0	0	0
61.0	14	6	0	0	0	0	0	0
62.0	12	5	0	0	0	0	0	0
63.0	10	4	0	0	0	0	0	0
64.0	8	3	0	0	0	0	0	0
65.0	6	2	0	0	0	0	0	0
66.0	5	2	0	0	0	0	0	0
67.0	3	1	0	0	0	0	0	0
68.0	2	1	0	0	0	0	0	0
69.0	2	0	0	0	0	0	0	0
70.0	1	0	0	0	0	0	0	0
71.0	0	0	0	0	0	0	0	0
72.0								

Table H.4.2 DISCHARGE THROUGH SPILLWAY AT KHADEJI DAM

H (m)	C	L (m)	Q (m^3/sec)	Water Level (m)
1.0	1.81	84	152	156.0
2.0	1.91	84	454	157.0
3.0	2.01	84	877	158.0
4.1	2.10	84	1,464	159.1
5.0	2.10	84	1,972	160.0
6.0	2.10	84	2,593	161.0
7.0	2.10	84	3,267	162.0
8.0	2.10	84	3,991	163.0
9.0	2.10	84	4,763	164.0
10.0	2.10	84	5,578	165.0

Remarks : $Q = C \times L \times H^{3/2}$

where, Q = Discharge (m^3/sec)
C = Avariable coefficent of discharge
L = Length of the crest = 84 (m)
H = Total head on the crest, including velocity of approach head

Table H.4.3 DISCHARGE THROUGH SPILLWAY AT MOL DAM

H (m)	C	L (m)	Q (m^3/sec)	Water Level (m)
1.0	1.81	320	589	172.8
2.0	1.97	320	1,783	173.8
3.0	2.08	320	3,459	174.8
3.4	2.10	320	4,213	175.2
4.0	2.10	320	5,376	175.8
5.0	2.10	320	7,513	176.8

Remarks : $Q = C \times L \times H^{3/2}$

where, Q = Discharge (m^3/sec)
C = Avariable coefficent of discharge
L = Length of the crest = 320 (m)
H = Total head on the crest, including velocity of approach head

Table H.4.4 CALCULATION OF FLOOD CONTROL FOR KHADEJI DAM (PMF)

Initial Water Level : 159.10 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)	Remarks
0	0	159.10	0	Gate Closed
:	:	:	:	Gate Partly Opened
200	1,464	159.10	1,464	Gate Fully Opened
380	5,002	161.20	2,728	
410	5,088	161.59	2,991	
420	5,117	161.71	3,072	
430	5,074	161.82	3,148	
560	4,528	162.71	3,782	
670	4,111	162.96	3,965	
680	4,071	162.97	3,970	
690	4,032	162.97	3,973	
700	3,993	162.98	3,974	
710	3,953	162.98	3,974	
720	3,913	162.97	3,973	
730	3,875	162.97	3,970	
740	3,835	162.97	3,966	
920	3,191	162.64	3,727	
1,100	2,687	162.07	3,314	
1,280	2,141	161.39	2,853	
1,460	1,459	160.50	2,285	
1,640	1,019	159.52	1,701	
1,725	881	159.10	1,464	
:	:	:	:	Gate Partly Closed
4,080	0	159.10	0	Gate Closed

Remarks:
1) Time of Maximum Water Level : 700 MIN
2) Maximum Water Level : 162.98 m
3) Maximum Inflow : 5,117 m*3/sec* (refer to Table H.4.1(1/2))
4) Maximum Outflow : 3,970 m*3/sec

Table H.4.5 CALCULATION OF FLOOD CONTROL FOR KHADEJI DAM (100 YEAR FLOOD)

Initial Water Level : 159.10 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)	Remarks
0	0	159.10	0	Gate Closed
:	:	:	:	Gate Partly Opened
270	1,464	159.10	1,464	Gate Fully Opened
450	2,182	159.69	1,799	
630	1,856	159.88	1,901	
810	1,506	159.64	1,768	
990	1,220	159.22	1,532	
1,040	1,156	159.10	1,464	
:	:	:	:	Gate Partly Closed
3,240	0	159.10	0	Gate Closed

Remarks:
1) Time of Maximum Water Level : 630 MIN
2) Maximum Water Level : 159.88 m
3) Maximum Inflow : 2,240 m*3/sec
4) Maximum Outflow : 1,901 m*3/sec

Table H.4.6 CALCULATION OF FLOOD CONTROL FOR KHADEJI DAM (50 YEAR FLOOD) (1)

Initial Water Level : 159.10 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)	Remarks
0	0	159.10	0	Gate Closed
:	:	:	:	Gate Partly Opened
300	1,464	159.10	1,464	Gate Fully Opened
360	1,810	159.22	1,530	
420	1,739	159.32	1,589	
480	1,629	159.36	1,611	
540	1,568	159.35	1,607	
600	1,457	159.31	1,585	
660	1,356	159.24	1,542	
720	1,259	159.14	1,486	
730	1,173	159.10	1,464	
:	:	:	:	Gate Partly Closed
3,120	0	159.10	0	Gate Closed

Remarks:
1) Time of Maximum Water Level : 480 MIN
2) Maximum Water Level : 159.36 m
3) Maximum Inflow : 1,820 m*3/sec
4) Maximum Outflow : 1,610 m*3/sec

Table H.4.7 CALCULATION OF FLOOD CONTROL FOR KHADEJI DAM (50 YEAR FLOOD) (2)

Initial Water Level : 155.00 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)	Remarks
0	0	155.00	0	Gate Closed
:	:	:	:	
430	1,798	159.10	1,464	Gate Partly Opened
490	1,704	159.22	1,532	
550	1,602	159.27	1,559	
610	1,539	159.27	1,563	
670	1,421	159.24	1,542	
730	1,324	159.17	1,501	
770	1,257	159.10	1,464	
:	:	:	:	Gate Partly Closed
3,120	0	159.10	0	Gate Closed

Remarks:
1) Time of Maximum Water Level : 510 MIN
2) Maximum Water Level : 159.28 m
3) Maximum Inflow : 1,820 m*3/sec
4) Maximum Outflow : 1,560 m*3/sec

Table H.4.8 CALCULATION OF FLOOD CONTROL FOR KHADEJI DAM (50 YEAR FLOOD) (3)

Initial Water Level : 149.00 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)	Remarks
0	0	149.00	0	Gate Closed
:	:	:	:	
660	1,582	159.10	1,464	Gate Partly Opened
720	1,498	159.14	1,485	
780	1,389	159.12	1,473	
800	1,385	159.10	1,464	
:	:	:	:	Gate Partly Closed
3,120	0	159.10	0	Gate Closed

Remarks:
1) Time of Maximum Water Level : 720 MIN
2) Maximum Water Level : 159.14 m
3) Maximum Inflow : 1,820 m*3/sec
4) Maximum Outflow : 1,490 m*3/sec

Table H.4.9 CALCULATION OF FLOOD CONTROL FOR MOL DAM (PMF)

Initial Water Level : 169.60 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)
0	0	169.60	0
180	119	169.67	41
360	2,256	171.21	1,314
540	4,085	172.70	3,655
590	4,248	172.90	4,015
600	4,280	172.92	4,066
610	4,239	172.94	4,106
620	4,199	172.96	4,129
630	4,158	172.96	4,139
640	4,117	172.96	4,138
650	4,077	172.96	4,130
660	4,036	172.95	4,115
720	4,096	172.93	4,083
900	3,685	172.77	3,782
1,080	3,182	172.50	3,296
1,260	2,775	172.26	2,881
1,440	2,244	171.96	2,388
1,620	1,696	171.63	1,836
1,800	1,292	171.30	1,426
1,980	1,002	171.03	1,098
2,160	791	170.83	863
2,340	556	170.63	625
2,520	378	170.42	482
2,700	233	170.16	329
2,880	140	169.95	206
3,060	83	169.81	125
3,240	48	169.73	74
3,420	29	169.67	44
3,600	17	169.64	26
3,780	10	169.63	15
3,960	5	169.61	8
4,140	2	169.61	4
4,320	0	169.60	1

Remarks:

1) Time of Maximum Water Level : 630 MIN
2) Maximum Water Level : 172.96 m
3) Maximum Inflow : 4,280 m*3/sec (refer to Table H.4.1(2/2))
4) Maximum Outflow : 4,140m:3/sec

Table H.4.10 CALCULATION OF FLOOD CONTROL FOR MOL DAM (100 YEAR FLOOD)

Initial Water Level: 169.60 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)
0	0	169.60	0
180	52	169.63	18
360	989	170.37	451
540	1,790	171.37	1,510
720	1,778	171.59	1,769
900	1,570	171.48	1,639
1,080	1,322	171.28	1,400
1,260	1,120	171.10	1,190
1,440	862	170.90	953
1,620	597	170.68	687
1,800	403	170.46	506
1,980	271	170.21	360
2,160	187	170.02	249
2,340	103	169.86	154
2,520	60	169.76	92
2,700	34	169.69	53
2,880	17	169.65	29
3,060	7	169.62	14
3,240	1	169.61	5
3,420	0	169.60	2

Remarks:
1) Time of Maximum Water Level: 720 MIN
2) Maximum Water Level: 171.59 m
3) Maximum Inflow: 1,872 m*3/sec
4) Maximum Outflow: 1,770 m*3/sec

Table H.4.11 CALCULATION OF FLOOD CONTROL FOR MOL DAM (50 YEAR FLOOD) (1)

Initial Water Level: 169.60 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)
0	0	169.60	0
180	43	169.62	14
360	804	170.22	368
540	1,453	171.13	1,216
720	1,437	171.31	1,432
900	1,259	171.21	1,317
1,080	1,049	171.04	1,115
1,260	876	170.89	935
1,440	658	170.72	734
1,620	440	170.52	543
1,800	292	170.26	389
1,980	198	170.05	266
2,160	138	169.91	183
2,340	74	169.79	113
2,520	42	169.71	66
2,700	23	169.66	37
2,880	11	169.63	20
3,060	4	169.62	9
3,240	0	169.60	3

Remarks:
1) Time of Maximum Water Level: 720 MIN
2) Maximum Water Level: 171.31 m
3) Maximum Inflow: 1,518 m*3/sec
4) Maximum Outflow: 1,430 m*3/sec

Table H.4.12 CALCULATION OF FLOOD CONTROL FOR MOL DAM (50 YEAR FLOOD) (2)

Initial Water Level: 163.00 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)
0	0	163.00	0
:	:	:	:
600	1,518	169.60	0
660	1,425	170.58	575
720	1,437	171.02	1,096
900	1,259	171.19	1,297
1,080	1,049	171.04	1,113
1,260	876	170.89	935
1,440	658	170.72	734
1,620	440	170.52	543
1,800	292	170.26	389
1,980	198	170.05	266
2,160	138	169.91	183
2,340	74	169.79	113
2,520	42	169.71	66
2,700	23	169.66	37
2,880	11	169.63	20
3,060	4	169.62	9
3,240	0	169.60	3

Remarks:
1) Time of Maximum Water Level: 840 MIN
2) Maximum Water Level: 171.21 m
3) Maximum Inflow: 1,518 m*3/sec
4) Maximum Outflow: 1,320 m*3/sec

Table H.4.13 CALCULATION OF FLOOD CONTROL FOR MOL DAM (50 YEAR FLOOD) (3)

Initial Water Level: 156.50 m

Duration (MIN)	Inflow (m*3/sec)	Water Level (m)	Outflow (m*3/sec)
0	0	156.50	0
:	:	:	:
750	1,410	169.60	0
900	1,259	171.03	1,106
1,080	1,049	171.03	1,102
1,260	876	170.89	934
1,440	658	170.72	734
1,620	440	170.52	543
1,800	292	170.26	389
1,980	198	170.05	266
2,160	138	169.91	183
2,340	74	169.79	113
2,520	42	169.71	66
2,700	23	169.66	37
2,880	11	169.63	20
3,060	4	169.62	9
3,240	0	169.60	3

Remarks:
1) Time of Maximum Water Level: 960 MIN
2) Maximum Water Level: 171.09 m
3) Maximum Inflow: 1,518 m*3/sec
4) Maximum Outflow: 1,170 m*3/sec

Table H.5.1 CALCULATION OF HEAD LOSS FOR OUTLET WORKS AT KHADEJI DAM

LOSS	ai (m^2)	fi	fi/ai^2	Remarks
1. Screen loss	1.767	0.39	0.125	Dia. 1.5m
2. Entrance loss	1.767	0.50	0.160	Dia. 1.5m
3. Friction loss	1.767	0.70	0.224	Dia. 1.5m*1
4. Gradual contraction loss	1.330	0.04	0.023	Dia. 1.3m
5. Gate loss(1)	1.330	0.20	0.114	Dia. 1.3m
6. Gate loss(2)	1.330	0.49	0.278	Dia. 1.3m
7. Exit loss	1.330	1.00	0.568	Dia. 1.3m
	Total	1.492		

$K=1/\sqrt{(fi/ai^2)}=1/\sqrt{(1.492)} = 0.819$

Remarks: ai : Section area of each part

fi : Coefficient of head loss

*1: $fi = 124.5 \times 0.013^2 \times 57/1.5^{4/3}$

Table H.6.1 CALCULATION OF HEAD LOSS FOR OUTLET WORKS AT MOL DAM

LOSS	ai (m^2)	fi	fi/ai^2	Remarks	
1. Screen loss	4.52	0.39	0.019	Dia. 2.4m	
2. Entrance (1) loss	4.52	0.50	0.024	Dia. 2.4m	
3. Friction (1) loss	4.52	0.39	0.019	Dia. 2.4m,	$\frac{124.5x0.013^2x60}{2.4^{4/3}}$
4. Exit (1) loss	4.52	1.00	0.049	Dia. 2.4m	
5. Entrance (2) loss	4.52	0.50	0.024	Dia. 2.4m	
6. Friction (2) loss	4.52	0.80	0.039	Dia. 2.4m,	$\frac{124.5x0.013^2x122}{2.4^{4/3}}$
7. Branch loss	4.52	0.03	0.001	Dia. 2.4m	
8. Gradual contraction loss	3.46	0.04	0.003	Dia. 2.1m	
9. Friction (3) loss	3.46	0.08	0.007	Dia. 2.1m,	$\frac{124.5x0.013^2x10}{2.4^{4/3}}$
10. Gate (1) loss	3.46	0.20	0.017	Dia. 2.1m	
11. Gate (2) loss	3.46	0.49	0.041	Dia. 2.1m	
12. Exit (2) loss	3.46	1.00	0.083	Dia. 2.1m	
	Σ		0.326		

$$K = \frac{1}{\sqrt{\Sigma(fc/ai^2)}} = \frac{1}{\sqrt{0.326}} = 1.751$$

Remarks: ai : Section area of each part

fi : Coefficient of head loss

FIGURES

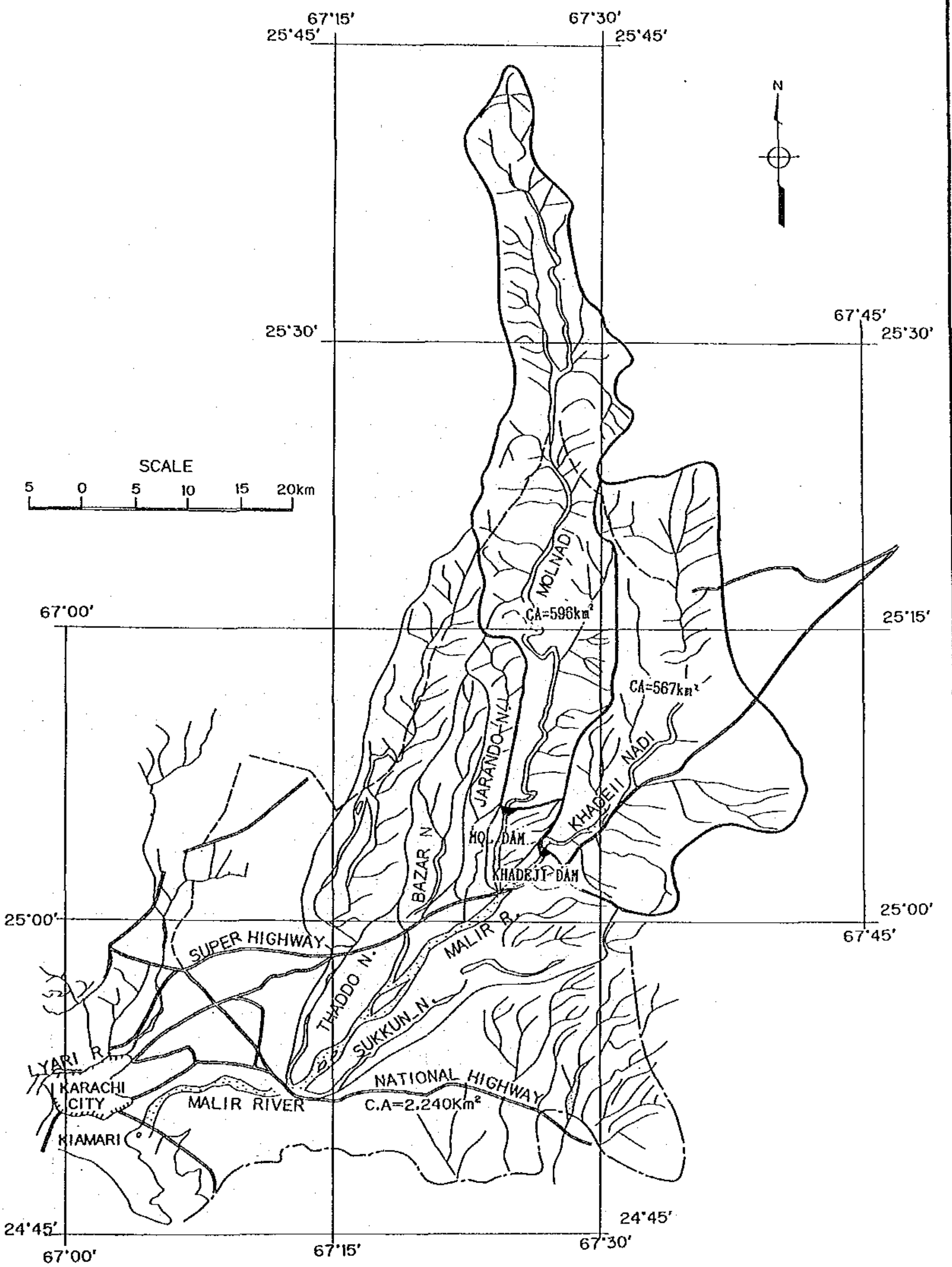

Fig. H.1-1 Location Map

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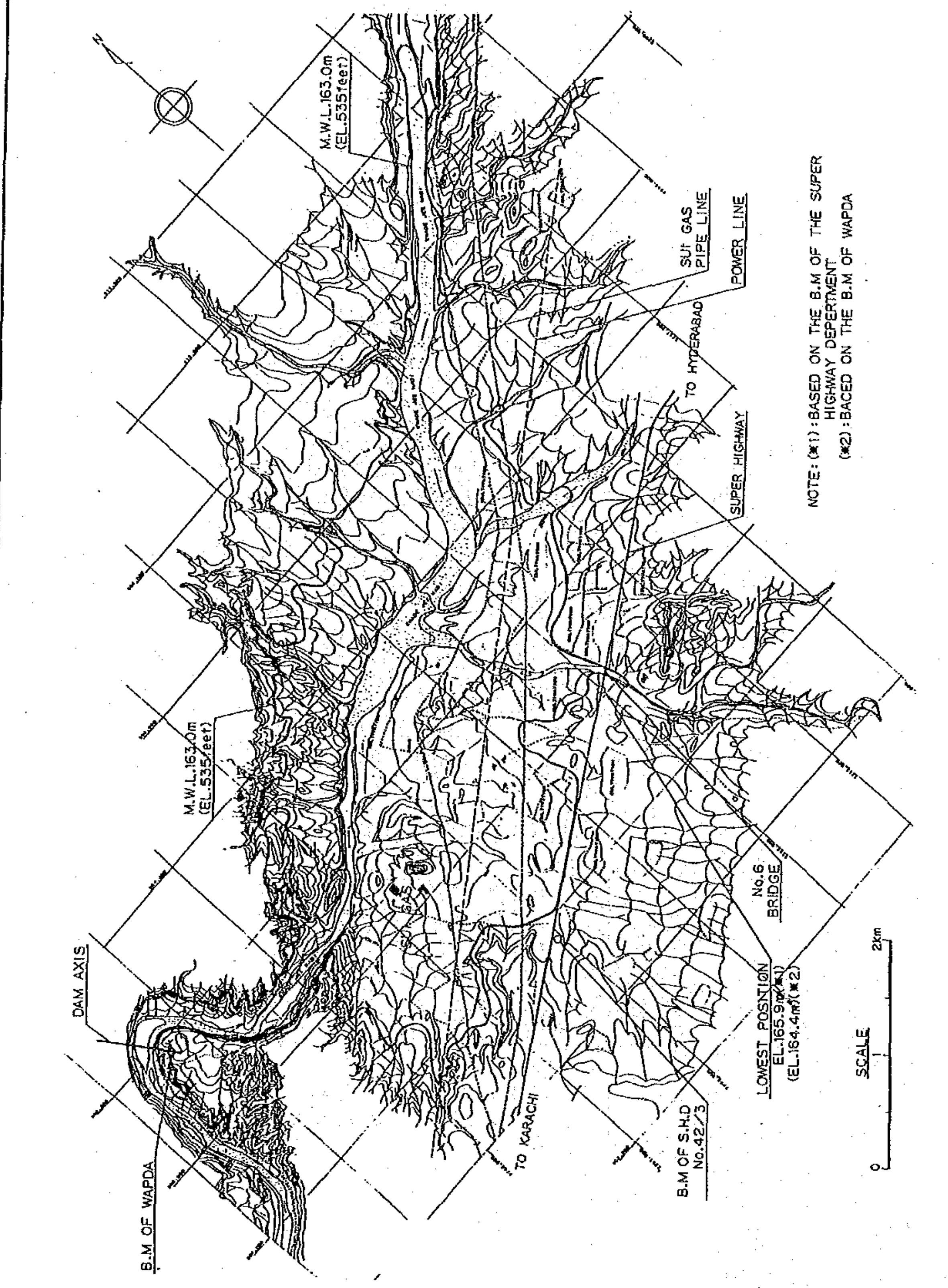

Fig. H.3-1 Reservoir Area of Khadeji Dam

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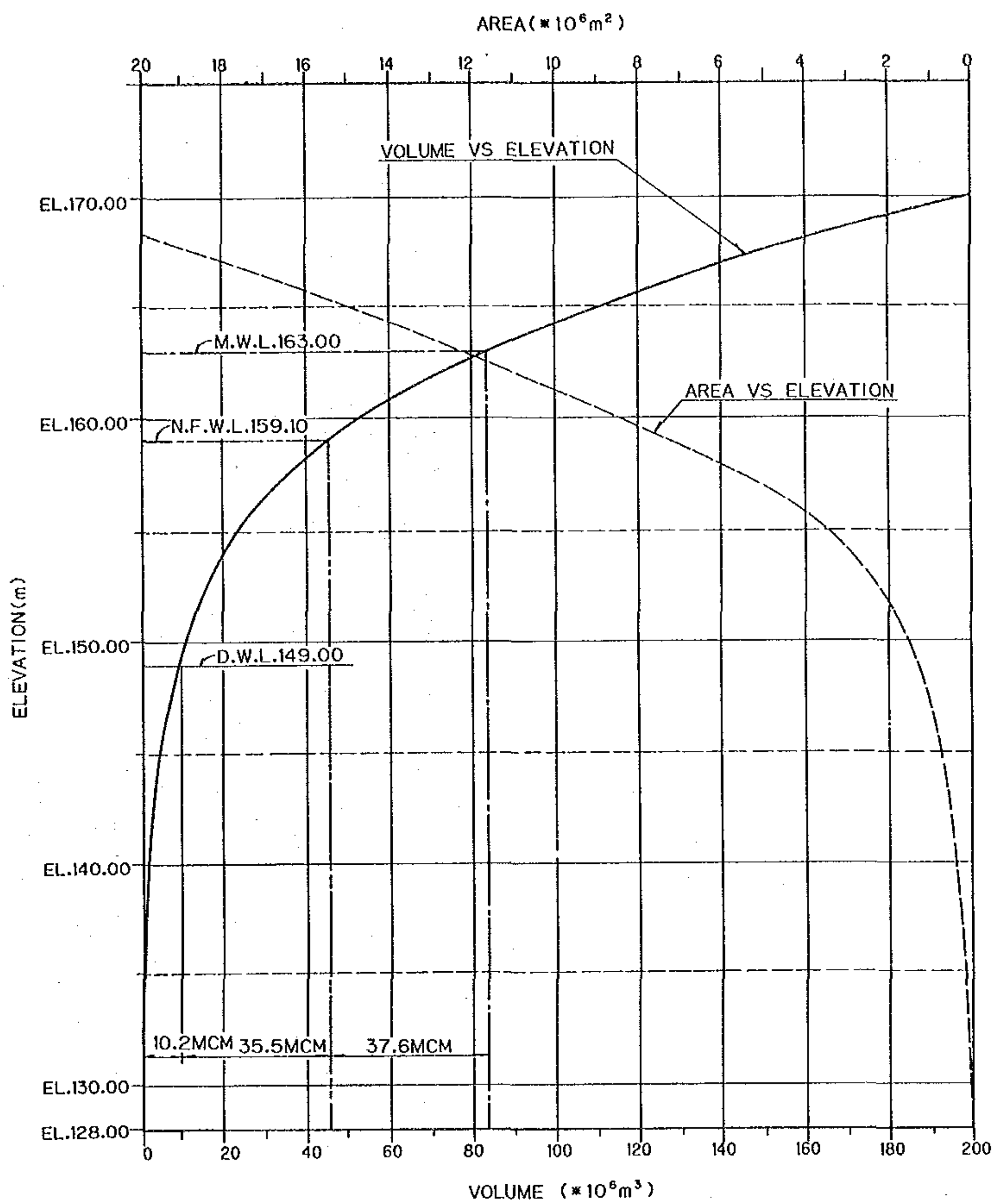

ELEVATION (m)	AREA (1,000 m2)	VOLUME (1,000 m3)
EL. 128.0	0	
EL. 140.2	382	2,330
EL. 146.3	843	6,070
EL. 152.4	2,119	15,100
EL. 158.5	6,477	41,320
EL. 164.6	14,354	104,550
EL. 170.7	23,230	218,880

Fig. H.3-2 Elevation-Area and Elevation-Storage Curves of Khadeji Dam

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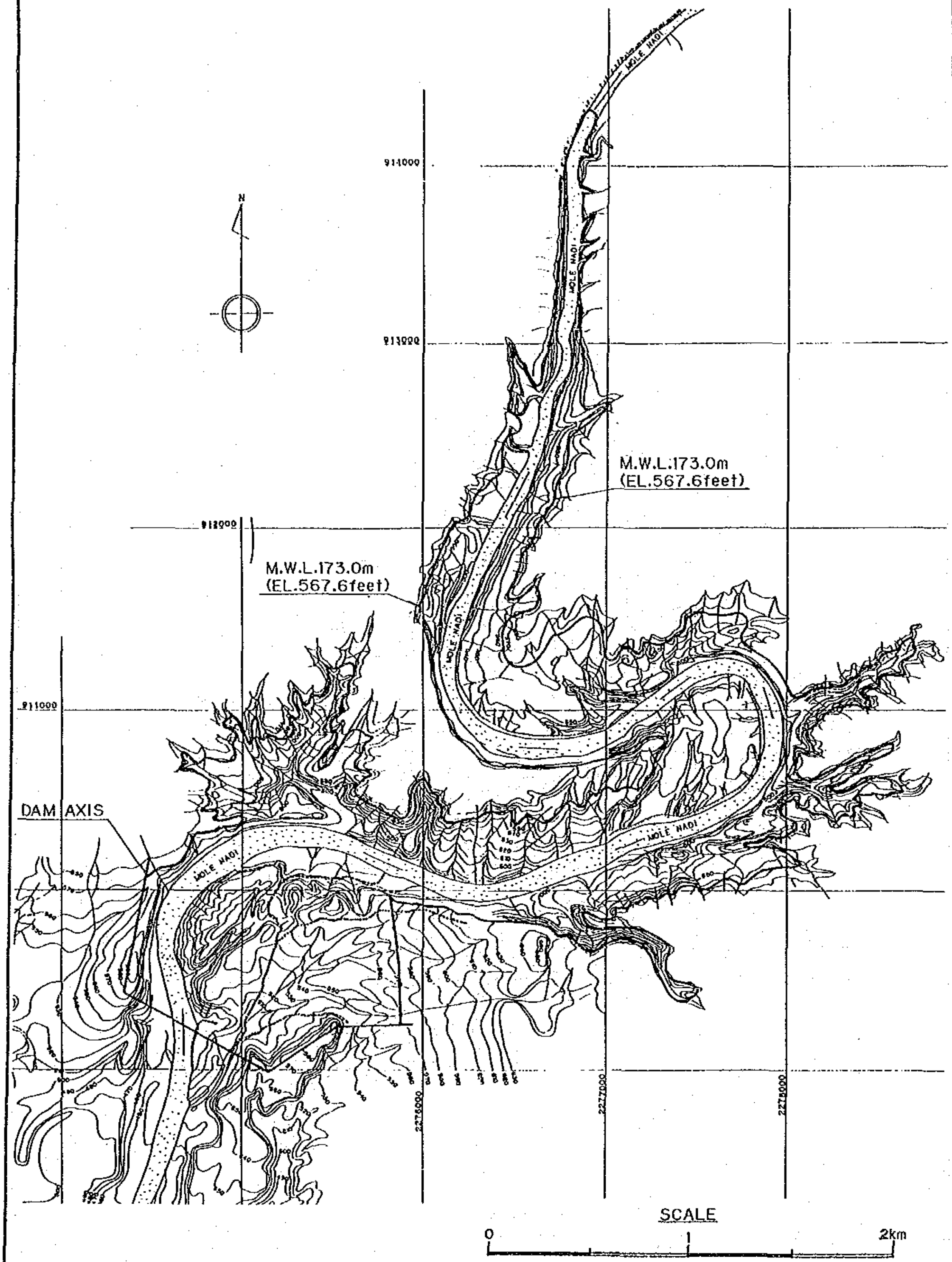

Fig. H.3-3 Reservoir Area of Mol Dam

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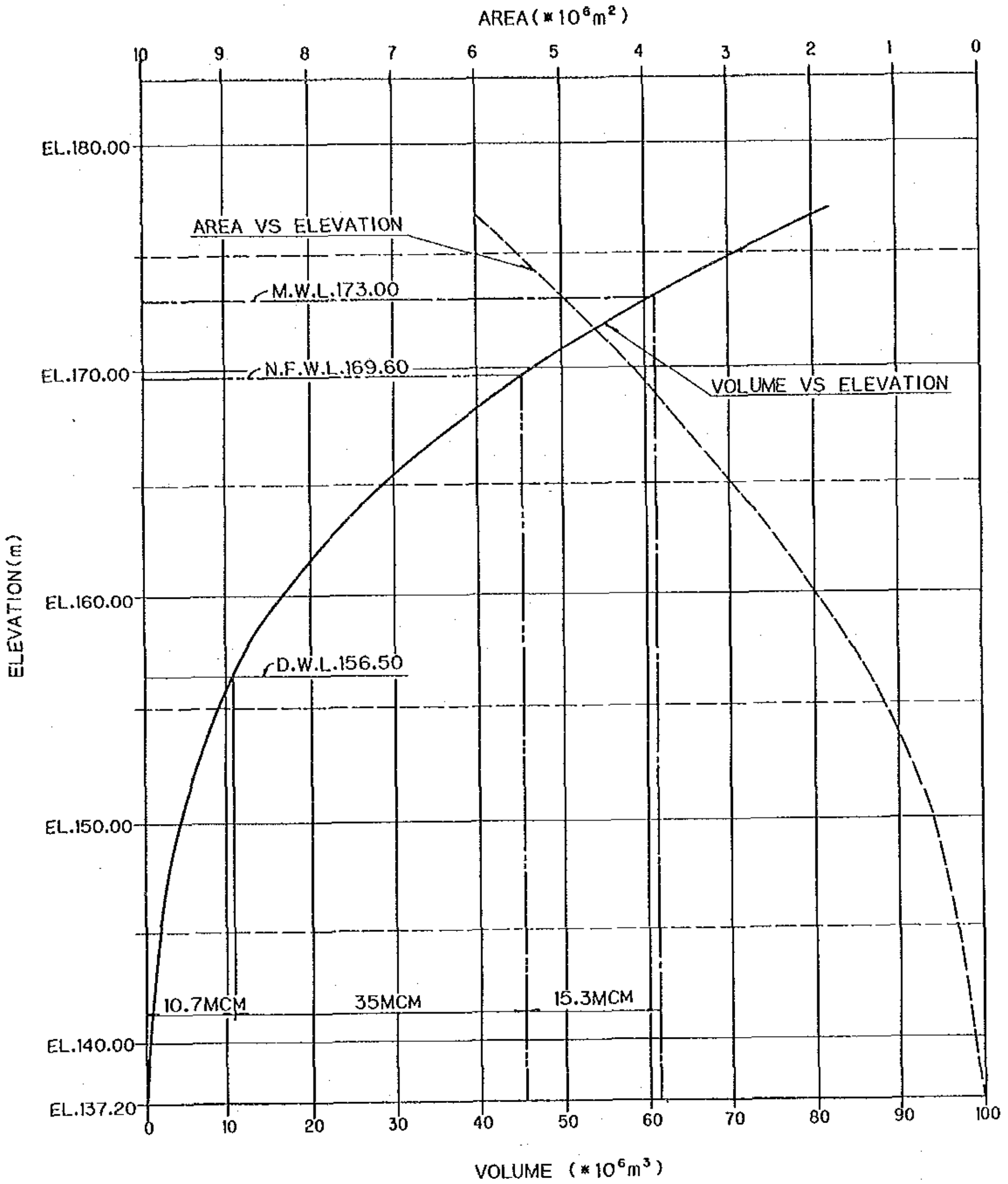

ELEVATION (m)	AREA (1,000 m2)	VOLUME (1,000 m3)
EL. 137.2	0	
EL. 143.3	303	920
EL. 149.4	596	3,660
EL. 154.4	1,232	9,150
EL. 161.5	2,295	19,900
EL. 167.6	3,565	37,780
EL. 173.7	5,212	64,540
EL. 176.8	5,958	81,860

Fig. H.3-4 Elevation-Area and Elevation-Storage Curves of Mol Dam

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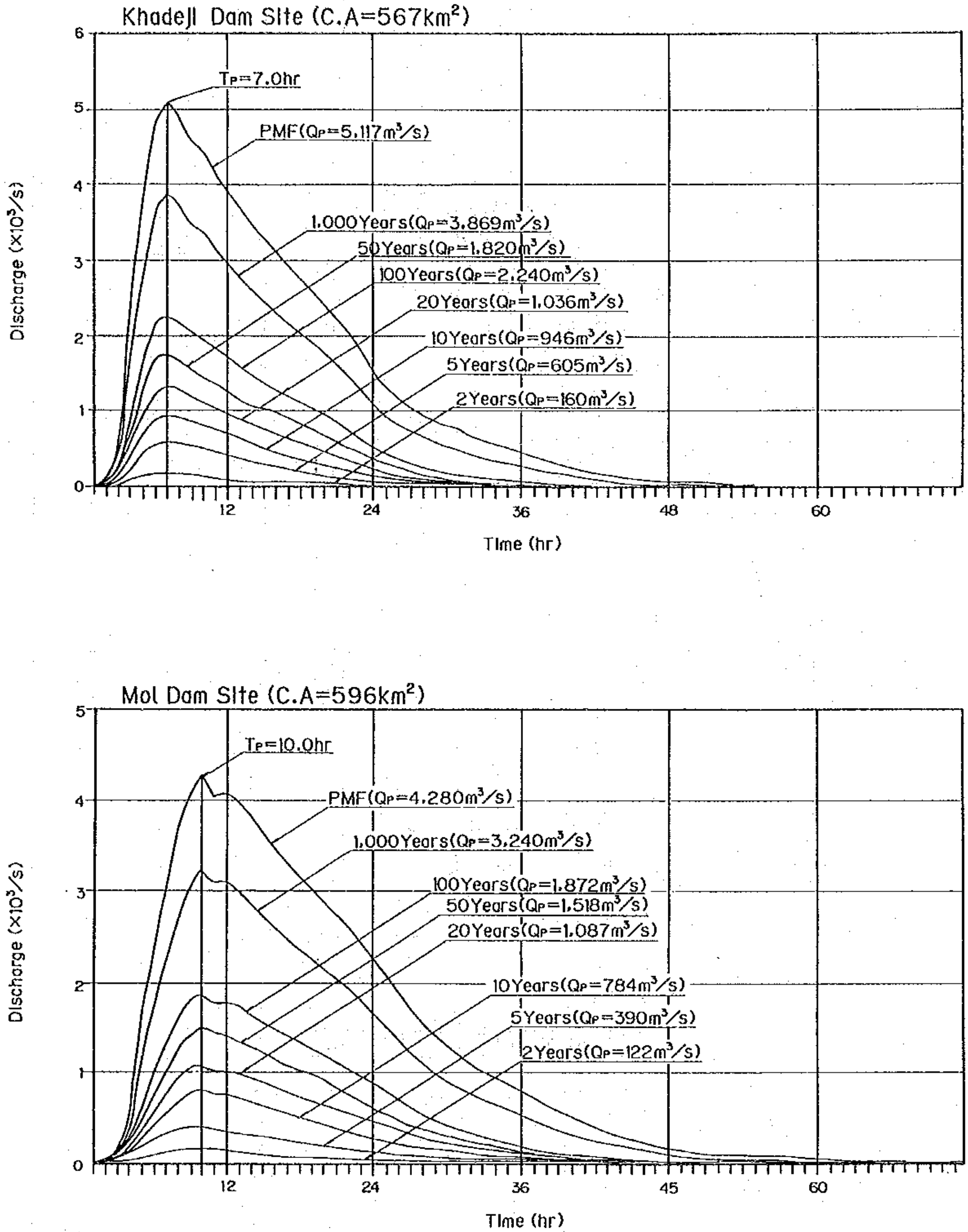

Fig. H.4-1 Probable Flood Hydrograph

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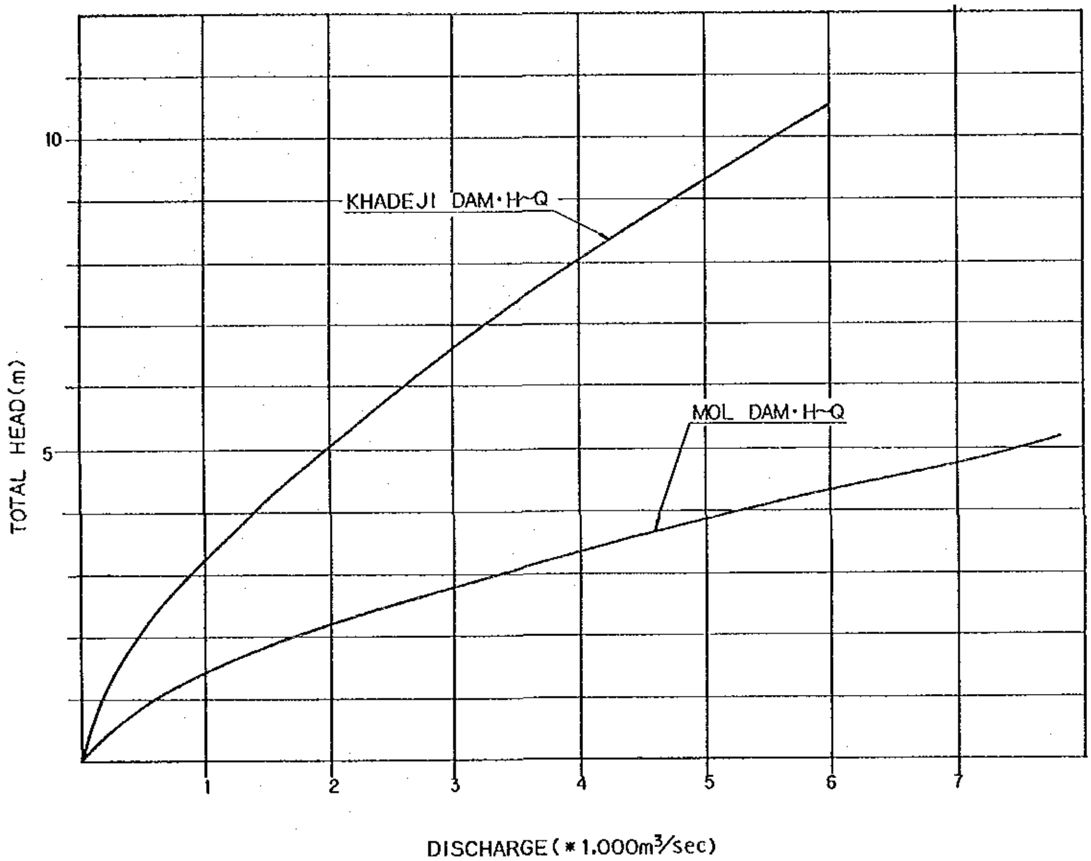

Fig. H.4-2 Spillway Design Discharges of Khadeji and Mol Dam

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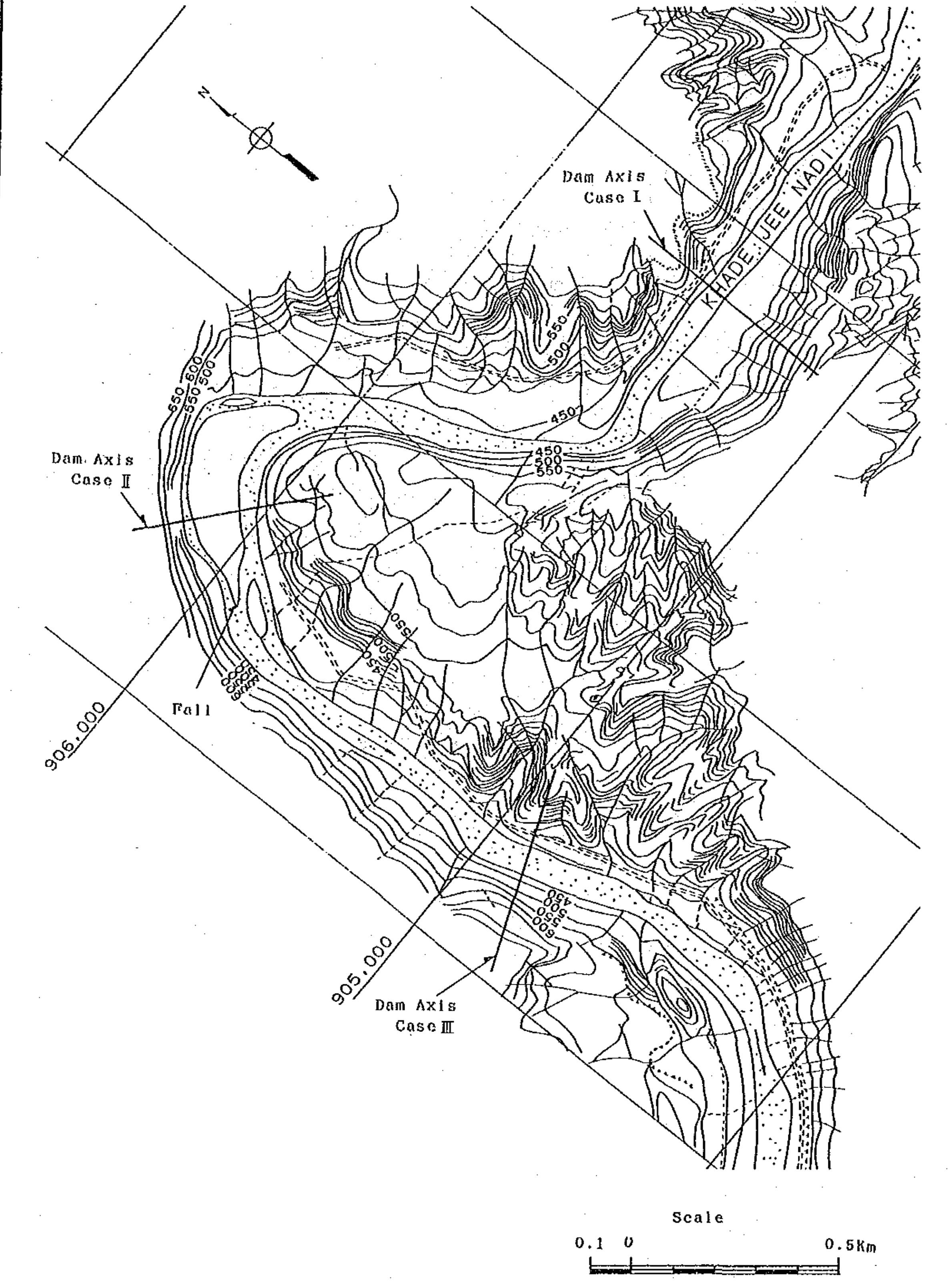

Fig. H.5-1 Dam Axis of Khadeji Dam

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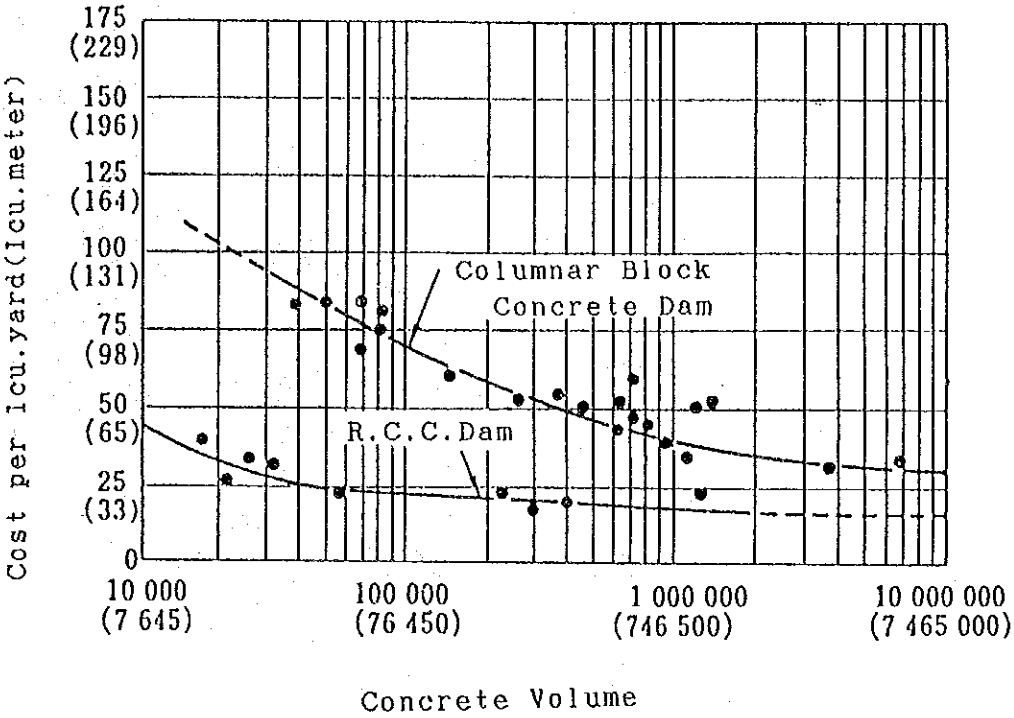

Fig. H.5-2 Comparsion of Construction Cost with R.C.C. and Columnar Block Method

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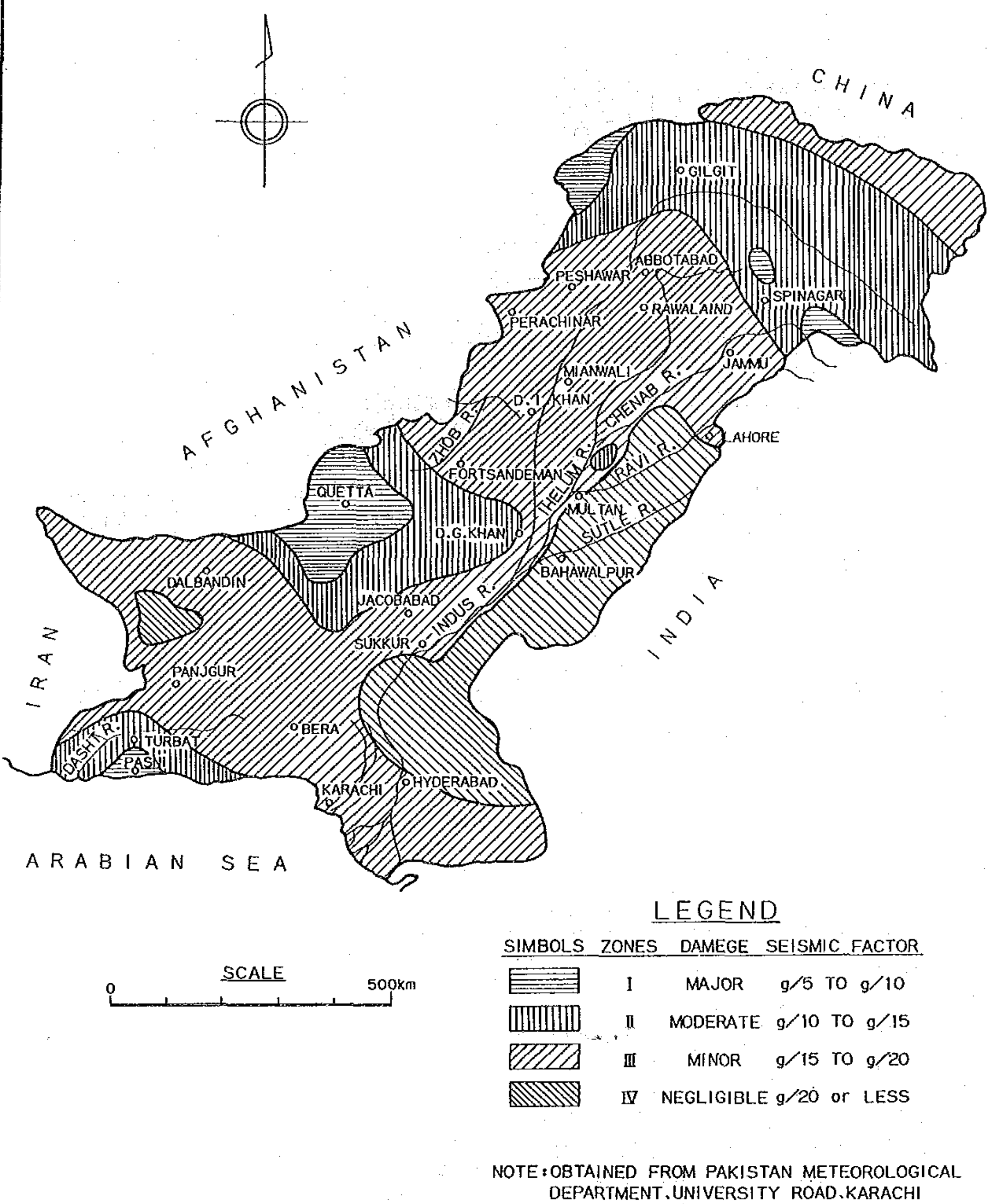

Fig. H.5-3 Seismic Zones in Pakistan

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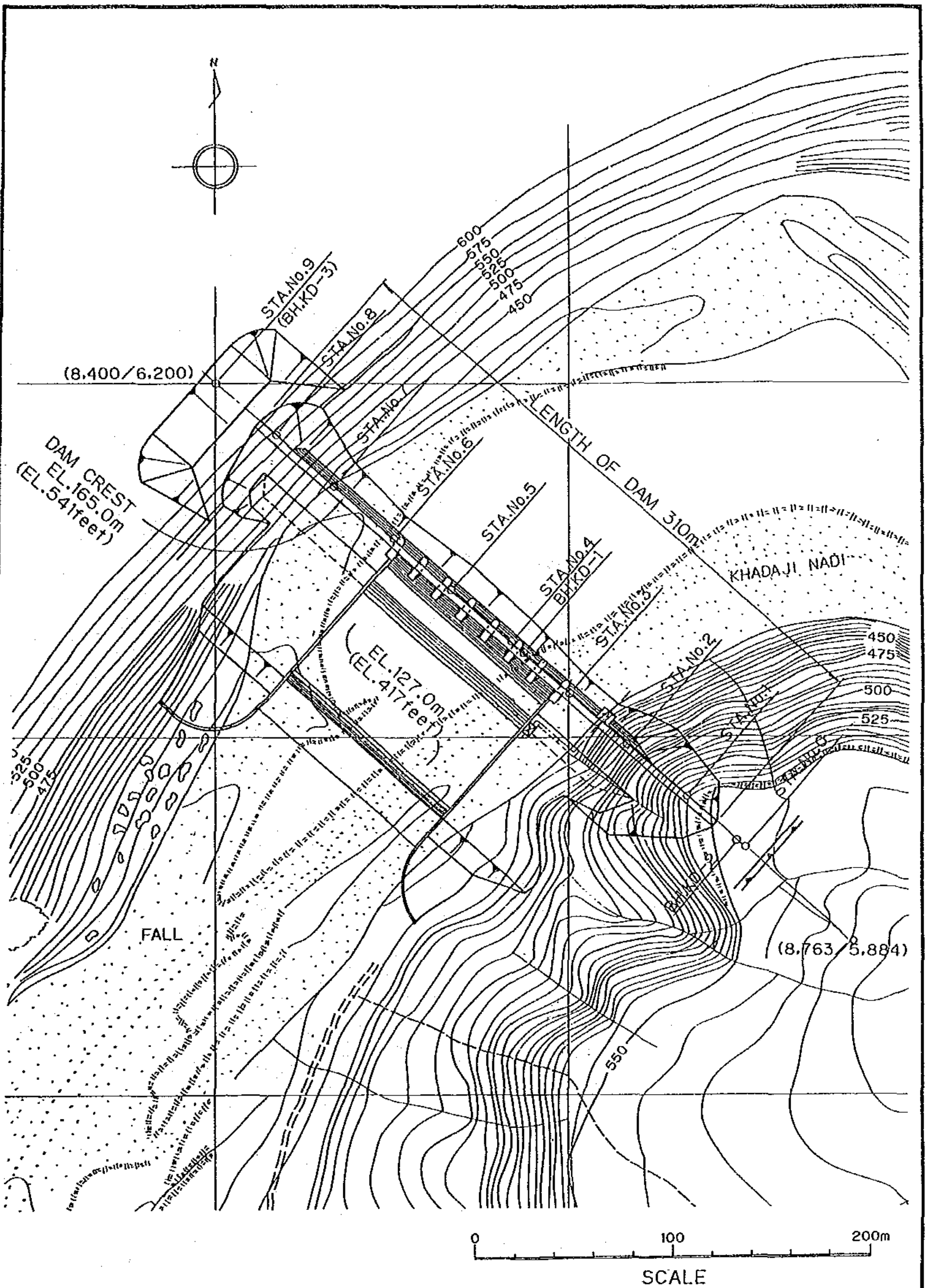

Fig.H.5-4 Plan of Khadeji Dam

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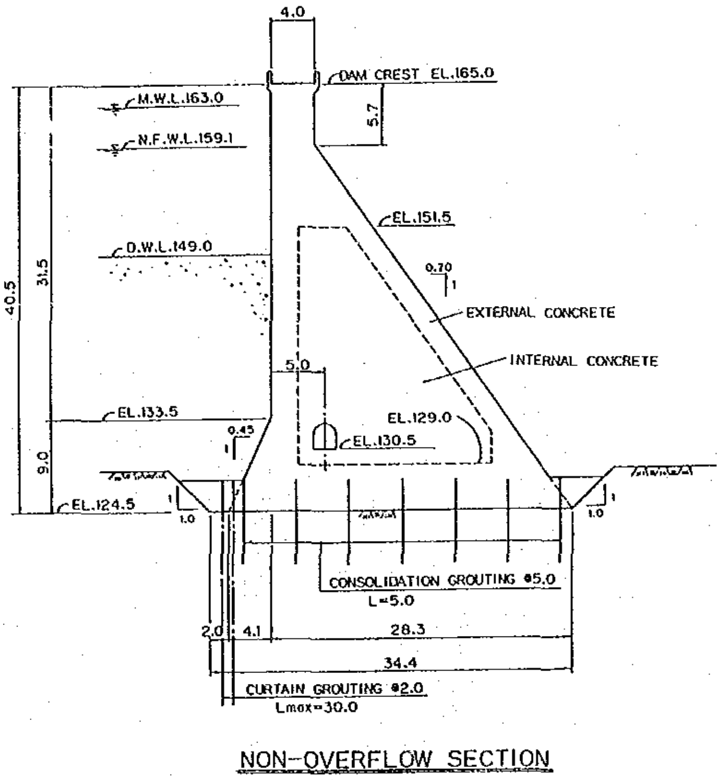

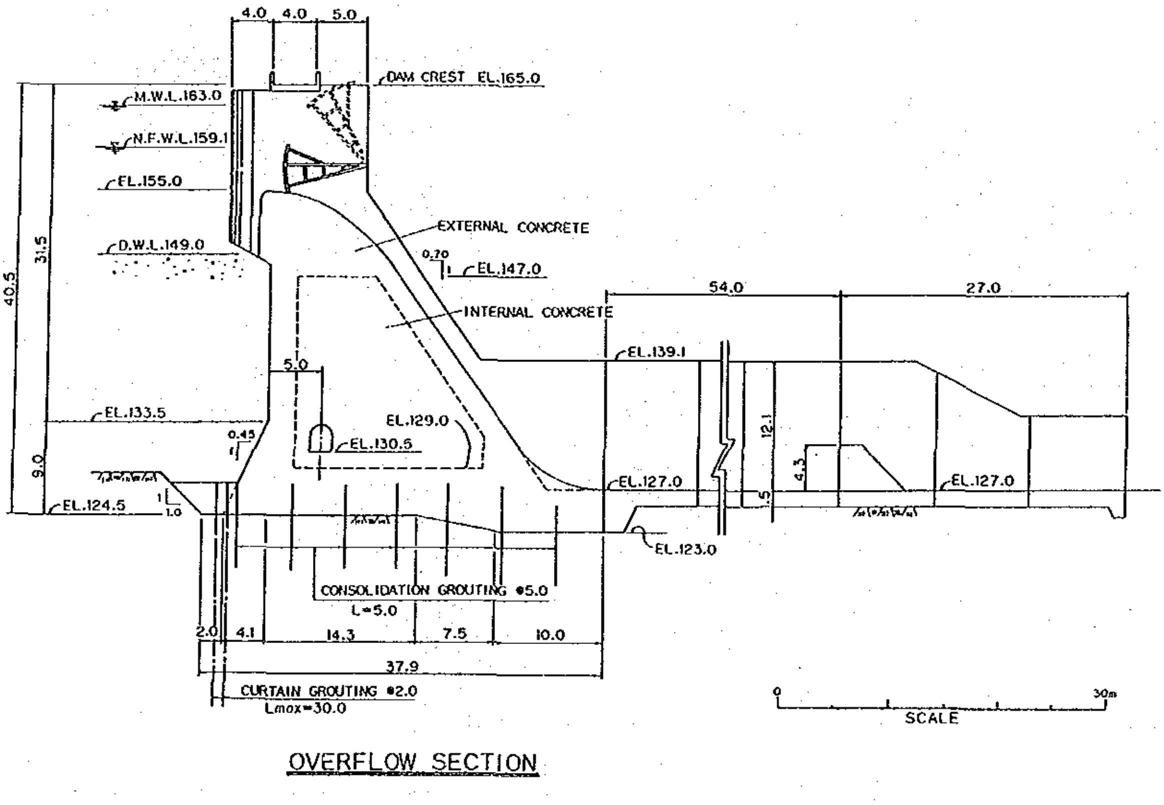

Fig.H.5-5 Standard Cross-Section of Khadeji Dam

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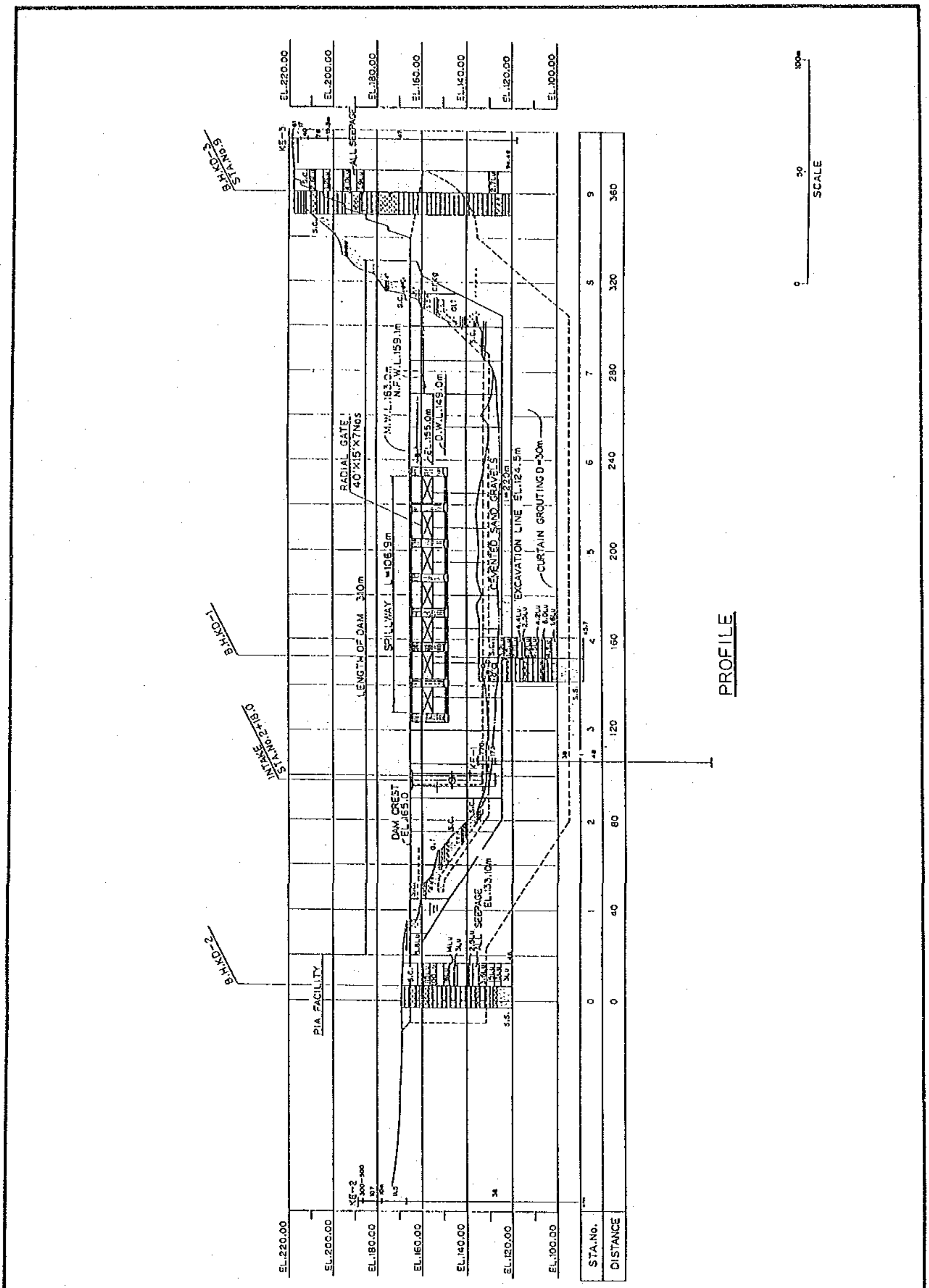

Fig.H.5-6 Profile of khadeji Dam

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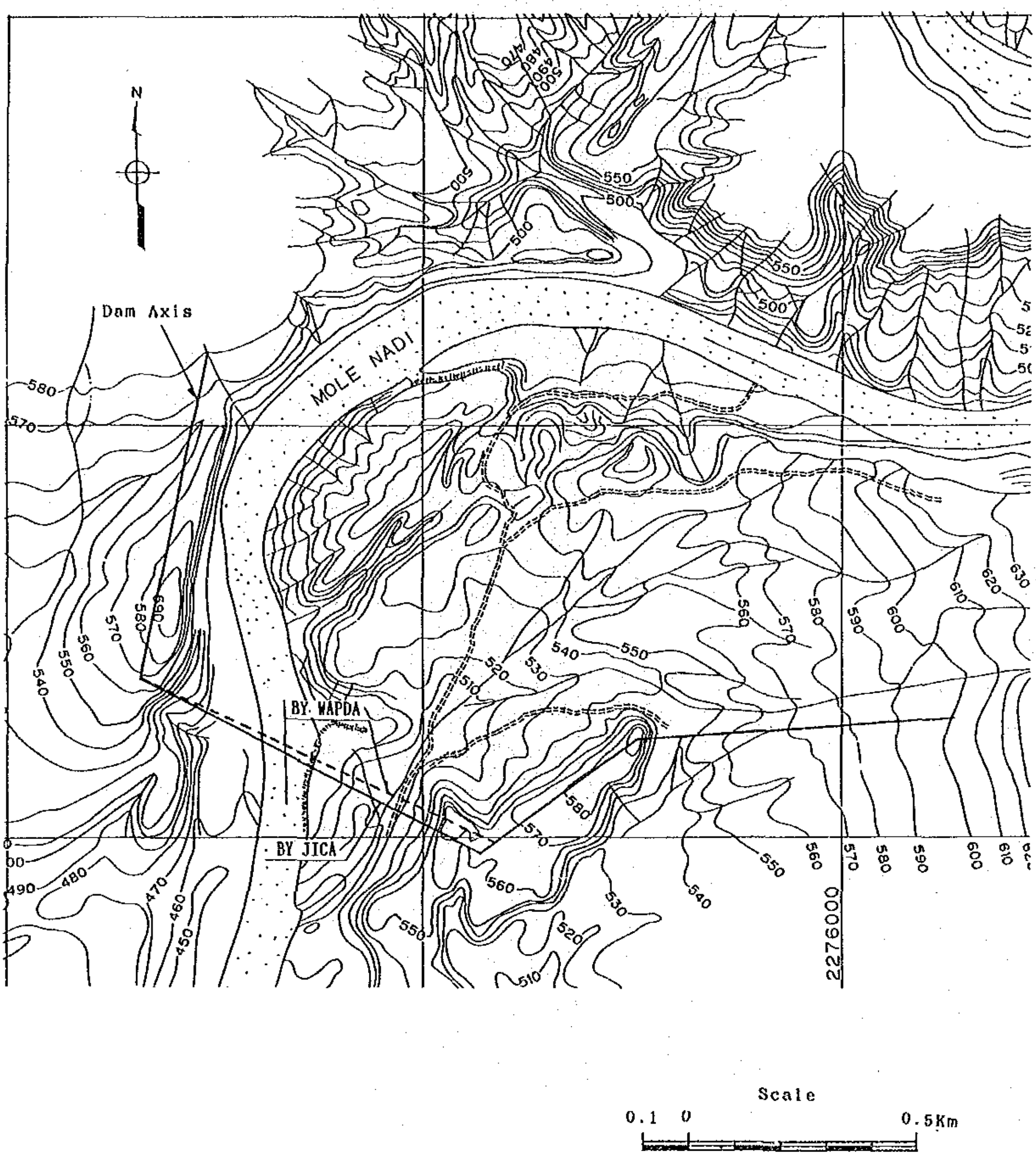

Fig. H.6-1 Dam Axis of Mol Dam

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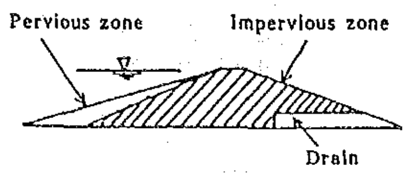
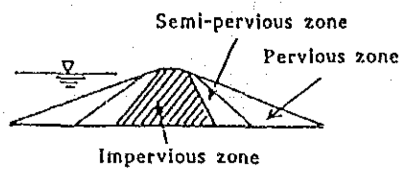
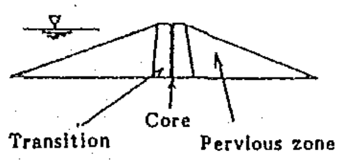
Dam type	Illustration	Definition
Homogeneous type		More than 80% of the maximum section of embankment is composed of homogeneous material
Zone type		A fill dam consisting of an inner or enclosed impervious earth material supported by outer zone of relatively pervious material
Facing type		Upstream slope of the dam is faced with impervious material other than soil
Core type		A fill dam provided by core which is composed of impervious material (asphalt, concrete, etc.) other than soil.

Fig.H.6-2 Classification of Fill Type Dam

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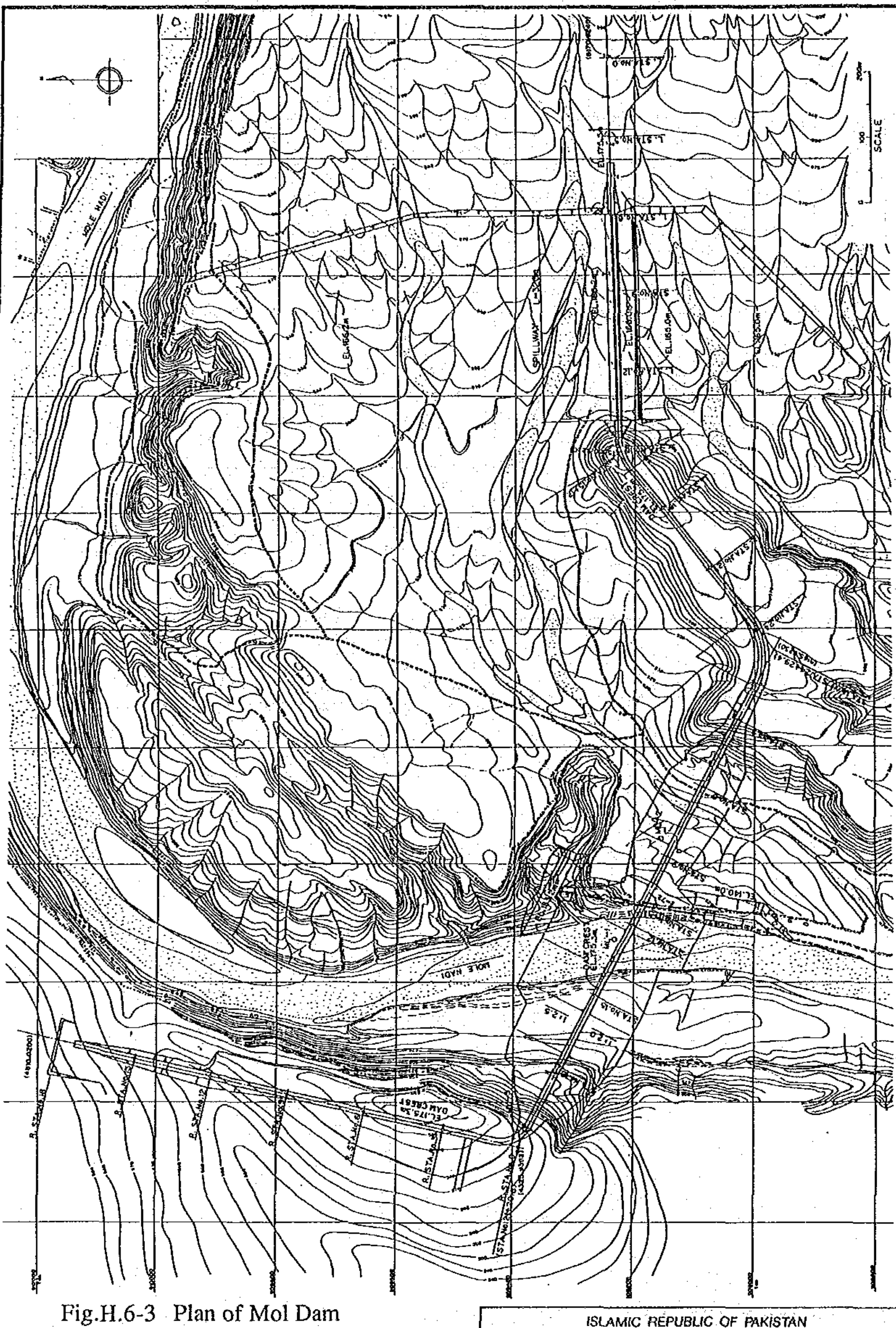

Fig.H.6-3 Plan of Mol Dam

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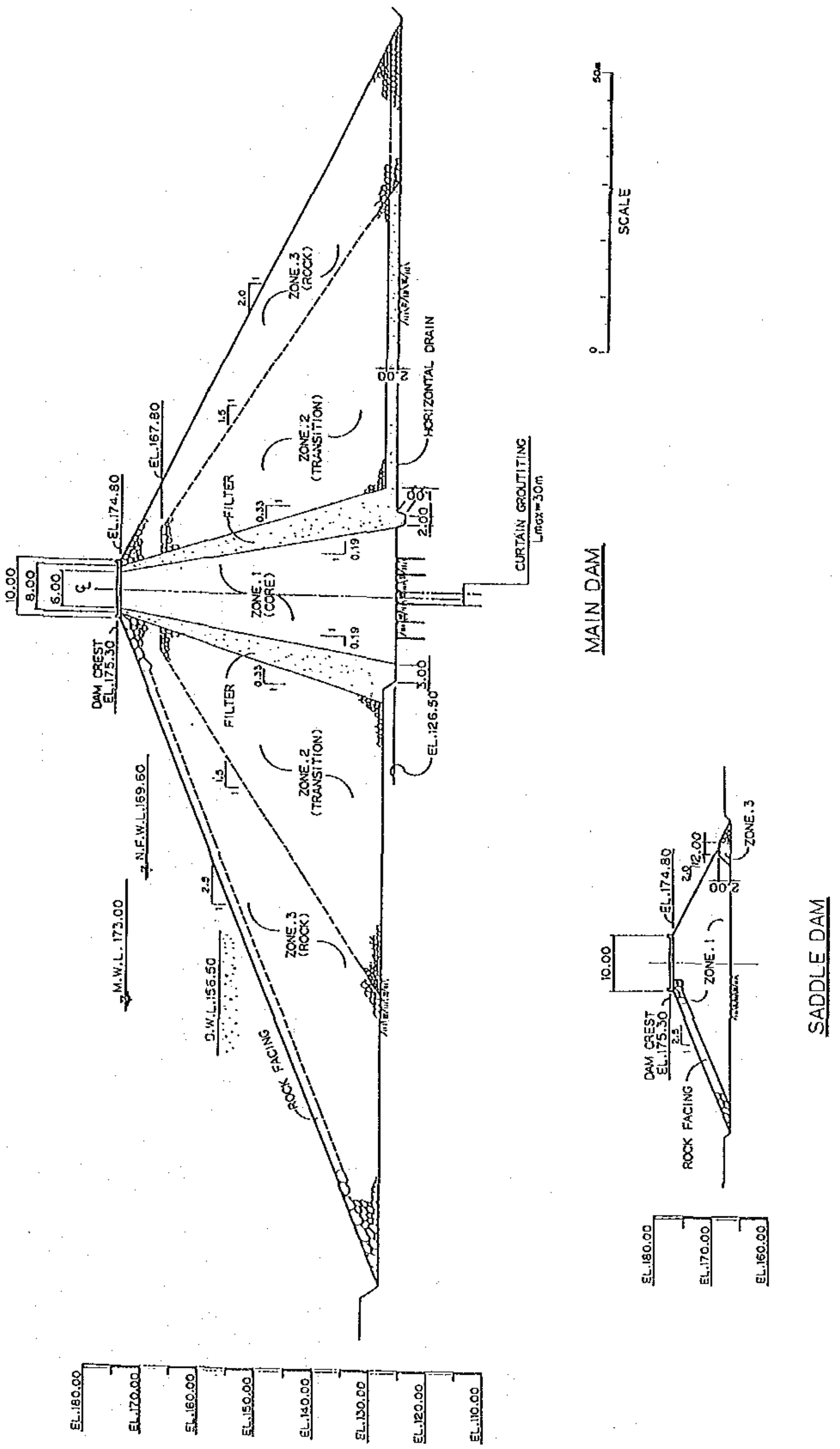

Fig.H.6-4 Standard Cross-section of Mol Dam

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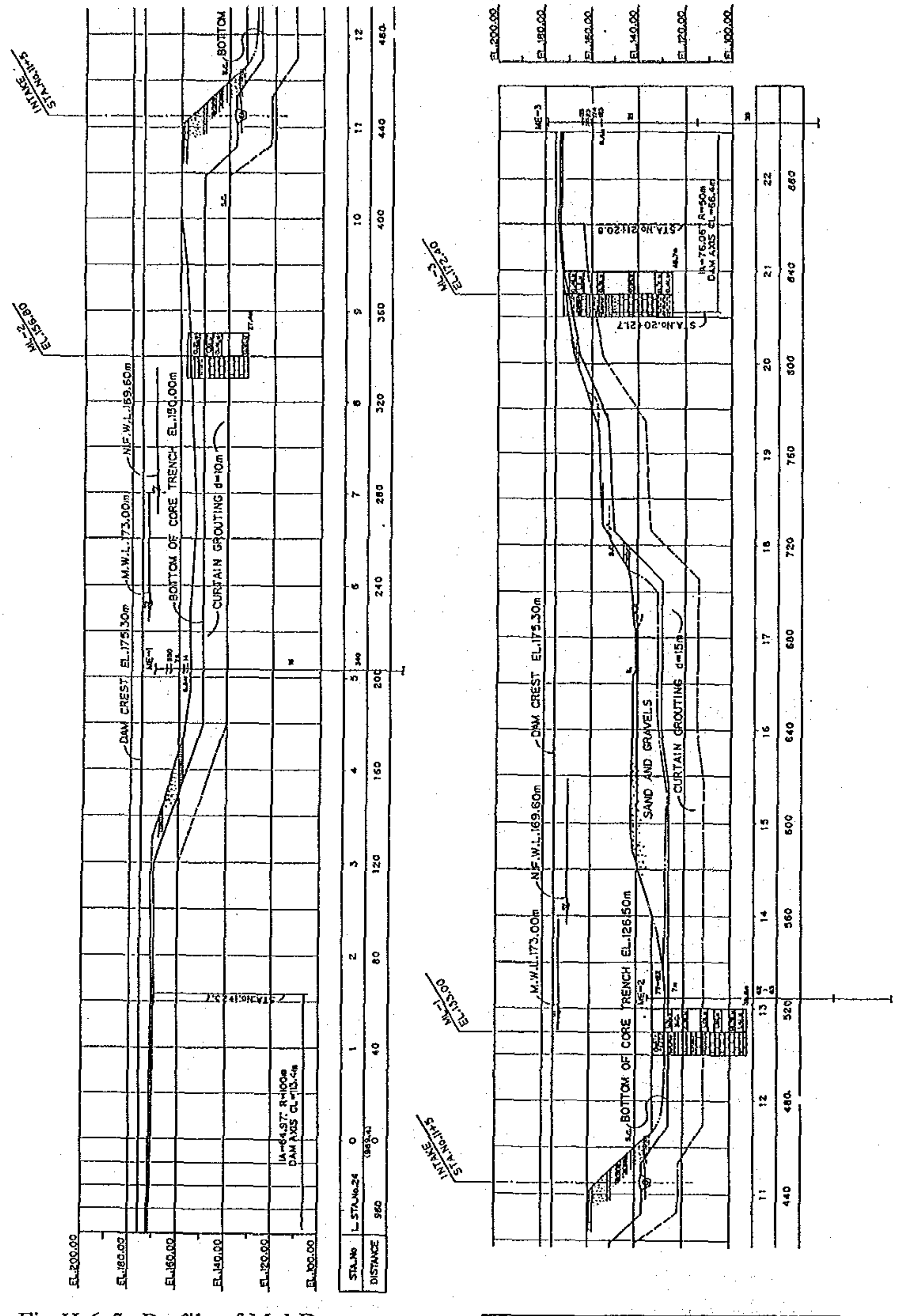

Fig.H.6-5 Profile of Mol Dam

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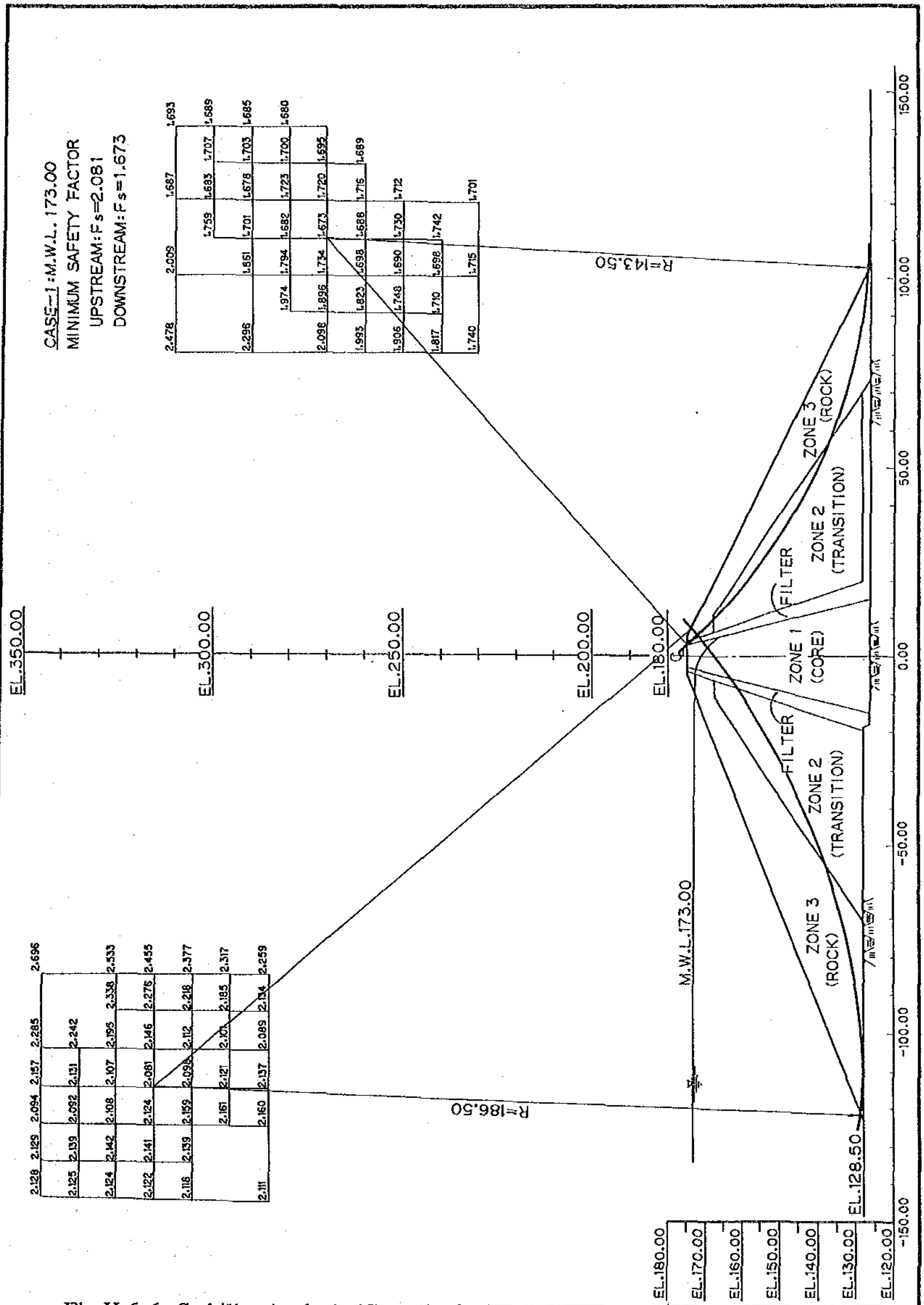

Fig.H.6-6 Stability Analysis (Case-1) of Mol Dam

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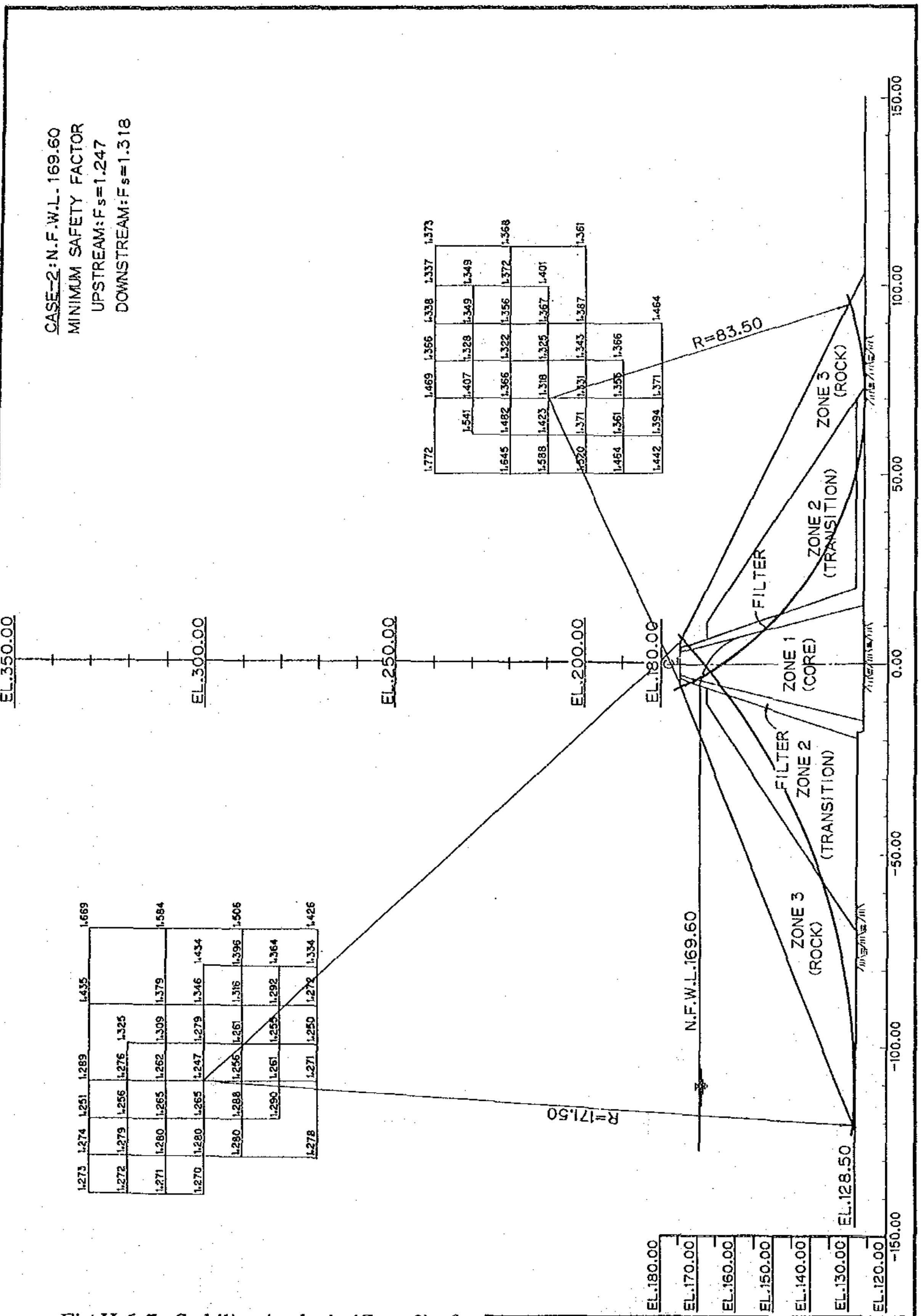

Fig.H.6-7 Stability Analysis (Case-2) of Mol Dam

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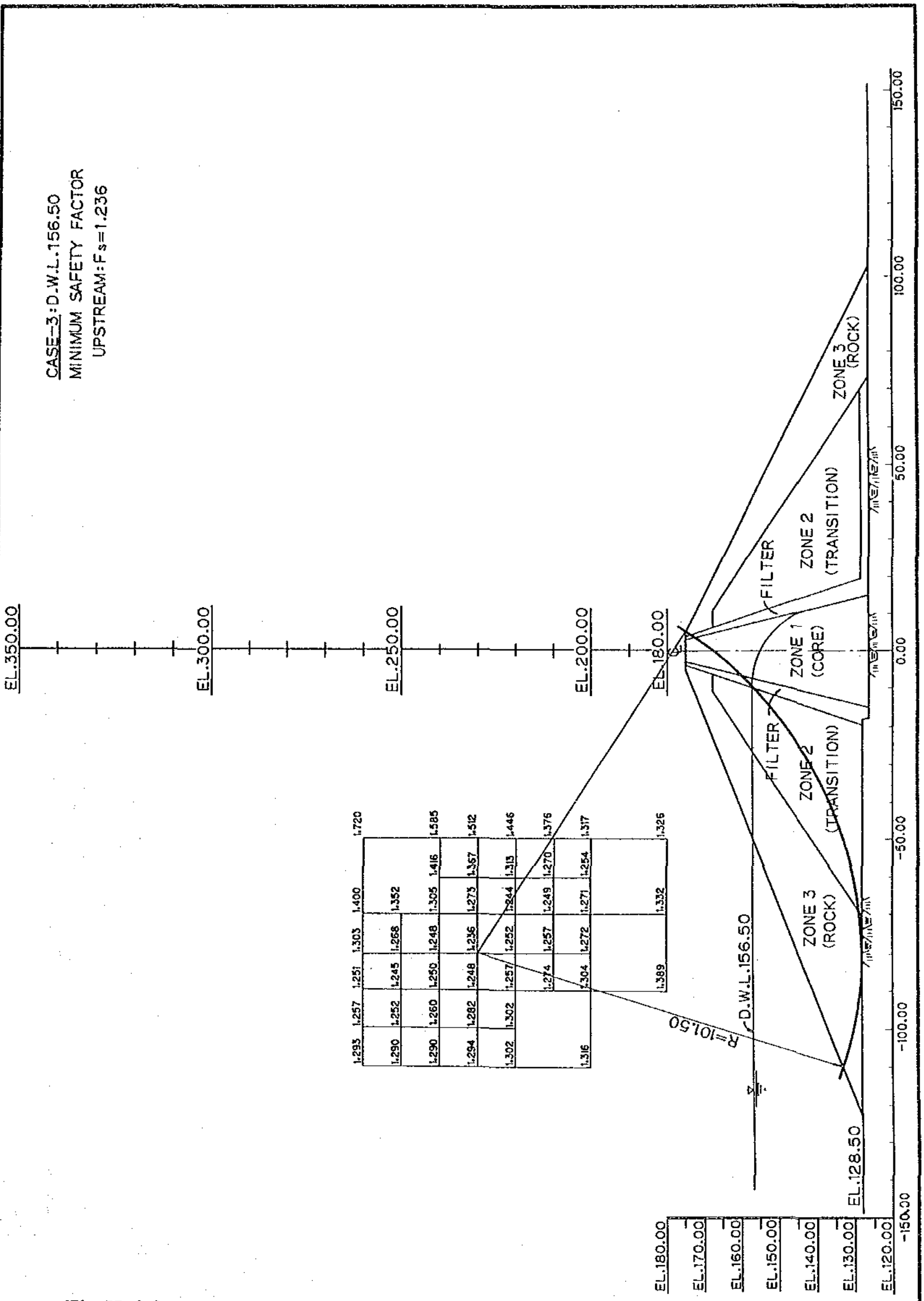

Fig.H.6-8 Stability Analysis (Case-3)of Mol Dam

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FEASIBILITY STUDY ON WATER RESOURCES DEVELOPMENT PROJECT IN THE MALIR BASIN
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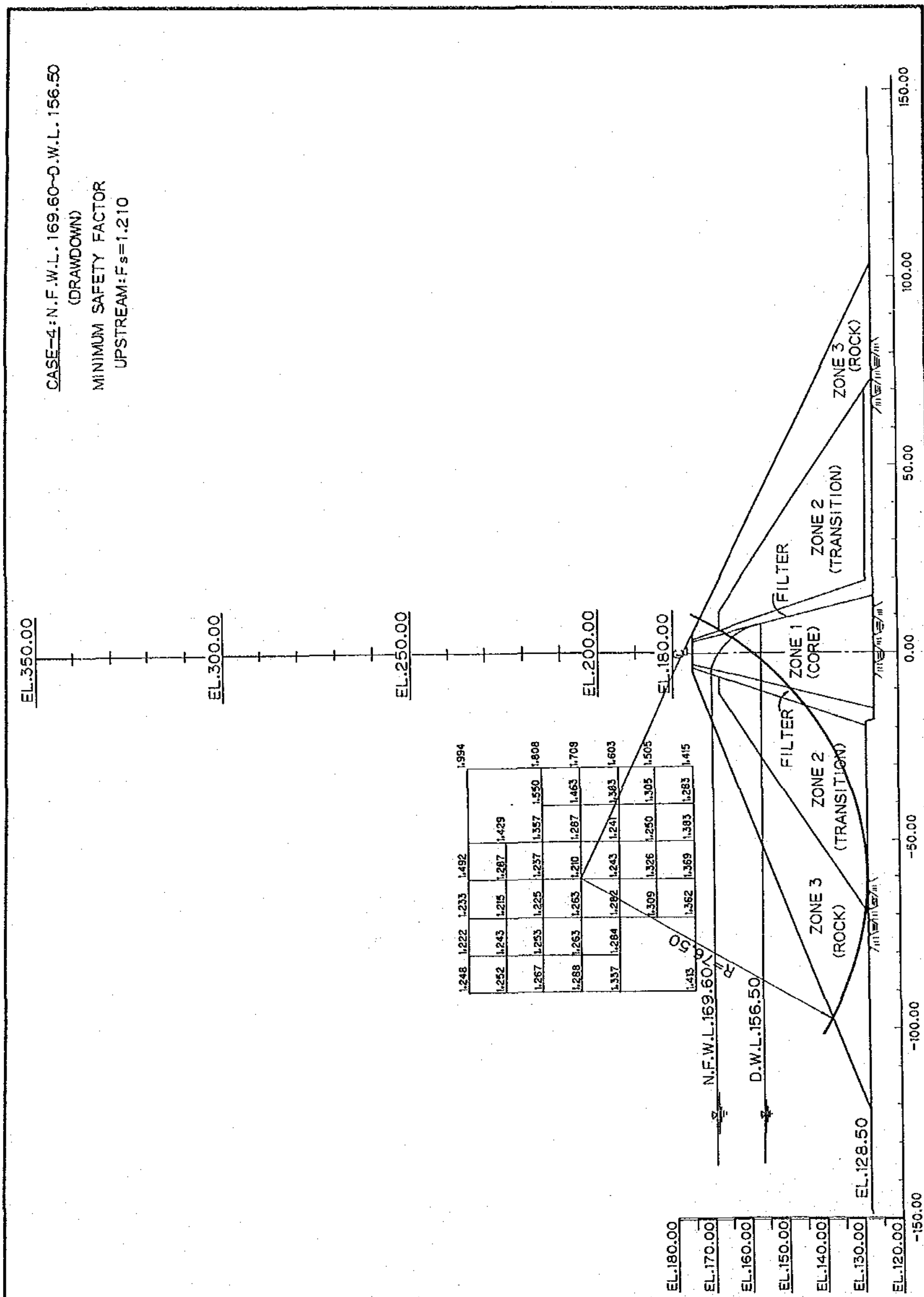

Fig.H.6-9 Stability Analysis (Case-4) of Mol Dam

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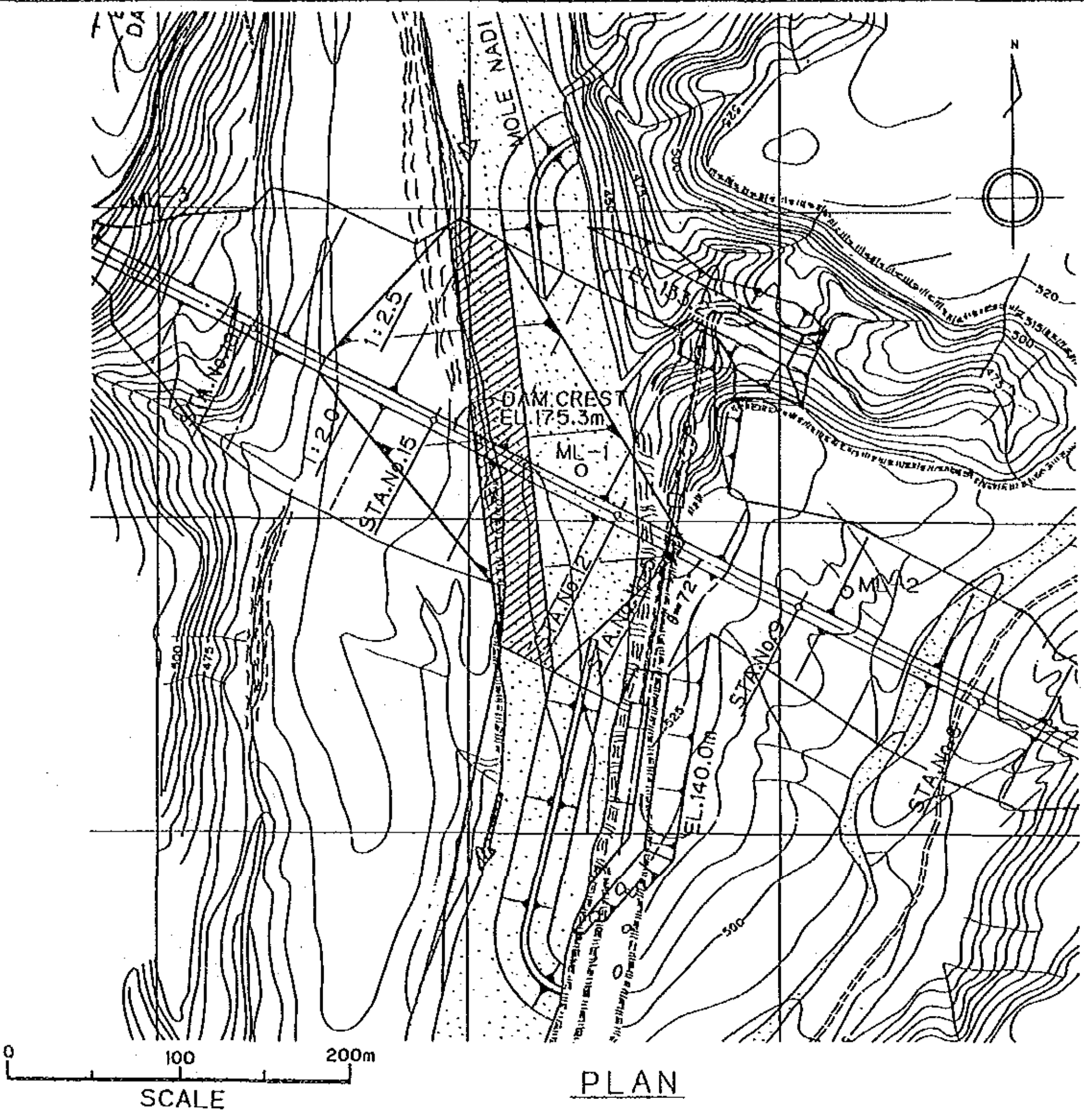

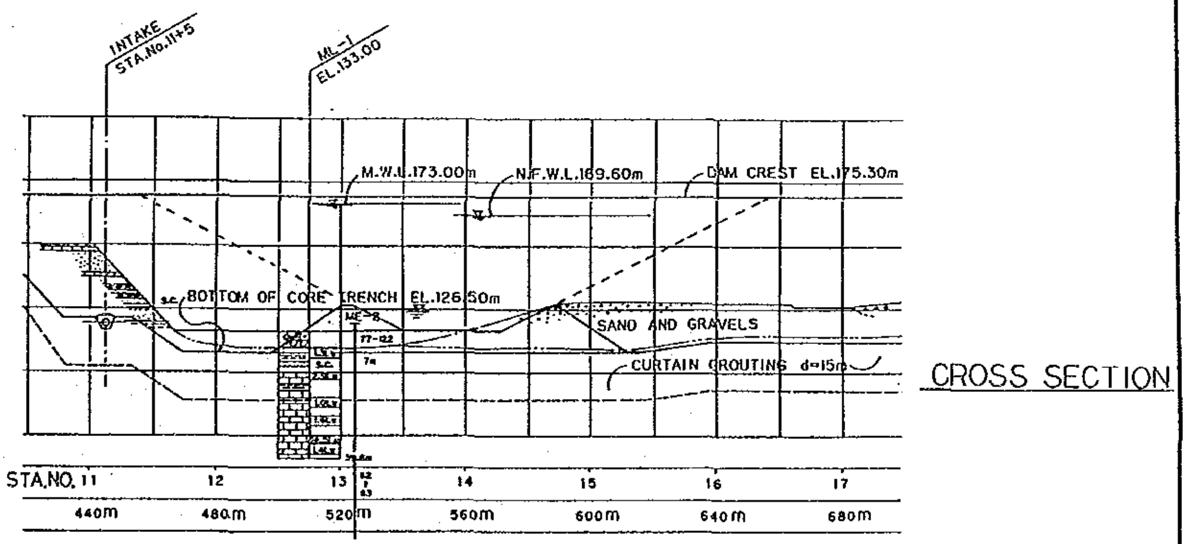

Fig.H.6-10 Temporary River Diversion Works at Mol Dam

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