# ***SECTOR C***

***PLAN FOR COMMUNITY POND,***

***FLOOD MITIGATION DAM AND DIVERSION CHANNEL***

# VOLUME 3: SUPPORTING REPORT

# SECTOR C: PLAN FOR COMMUNITY POND, FLOOD MITIGATION DAM AND DIVERSION CHANNEL

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# SECTOR C PLAN FOR COMMUNITY POND, FLOOD MITIGATION DAM AND DIVERSION CHANNEL

## 1. POTENTIAL STRUCTURAL FLOOD MITIGATION MEASURES

As the results of field reconnaissance, interview survey on the extent of the past floods, and review on the previous relevant studies, the followings are preliminarily scrutinized as the potential structural measures for flood mitigation of Lai Nullah (refer to Fig. C.1):

(1) Community pond at Fatima Jinnah Park in Islamabad;

(2) Flood mitigation dam to be placed in the area administratively called Block E-11 of Islamabad;

(3) Flood diversion channel to divert the flood discharge from tributaries of Bedarawali Kas, Tenawali Kas and Saidpur Kas to Kurang river;

Concepts of the above potential structural measures are as described in the followings.

### 1.1 Community Pond

#### 1.1.1 Possible Site

A community pond has the function of temporarily storing runoff discharge on the way to the upper or middle reaches of a river and thus flattening the peak runoff discharge. This measure is very effective for the control of flood with a short flood concentration time and it is technically managed to detain runoff discharge before joining into the lower rivers that do not have sufficient flow capacities. However, its applicability definitely depends on a suitable site that technically and regionally allows temporary inundation, because this type of facility requires large flood regulation capacity and a rather extensive land acquisition.

In the study area, possible sites for community pond are very limited. In Rawalpindi, the land along the river course is fully and disorderly utilized as built-up area with dense population. In Islamabad, urbanization has been neatly promoted in the form of square lots, each of which is used for a specific purpose such as administration, commercial and residential areas.

Under the above land use conditions in the study area, the Fatima Jinnah Park covering an extent of 3 $km^2$ located in the north of the study area is a strong candidate of the site for the community pond (refer to Fig. C.2). It was planned and constructed at administratively called Block F-9 as the National Park in the capital city in 1960's. The substantial part of it is still remained as the vacant land without any major permanent structure (refer to Fig. C.3).

Taking the above into consideration, the community pond is proposed to construct at the Fatima Jinnah Park. The principal advantages of the proposed pond are as enumerated below:

(1) Any land acquisition and house evacuation is not required,

(2) CDA, the administrator of the park has given the provisional consent to use the park as the flood detention facilities in view of the function of community pond to improve the amenity of the park,

(3) The flood retarding basin with the function of community pond can widely produce the benefits such as leading to effective land use, lowering of land development cost and creating of the urban scenery through introduction of greening and water-based beautification.

### 1.1.2 Flood Diversion from Tributary of Bedarawali Kas

The catchment area of the community pond proposed on the tributary of Tenawali Kas is about 16.6 $km^2$, which is equivalent to only 7 % of the Lai Nullah River Basin. Generally, the larger catchment area brings out the higher effectiveness of the flood control function and the greater the cost advantage of the facilities. From this point of view, it is proposed to divert the flood discharge of Bedarawali Kas to the community pond as shown in the following figure.

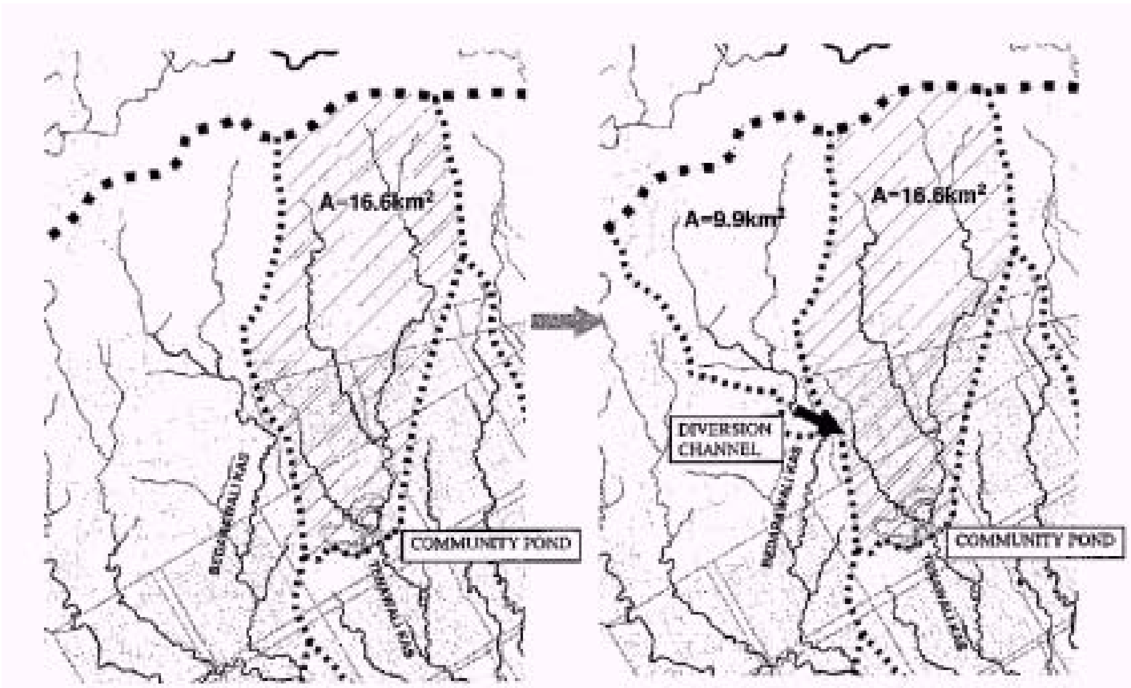


Fig. R C.1 Flood Diversion from Tributary of Bedarawali Kas to Community Pond

The total catchment area of the community pond becomes about 26.5 $km^2$, which is equivalent to 11.3 % of the Lai Nullah River Basin. The length of the diversion channel is about 1,340 m (refer to Fig. C.4).

### 1.1.3 Layout of Community Pond

The community pond should be designed hydraulically to have the flood control function as described in the previous section. In the Fatima Jinnah Park, small dam with flood control function is planed on the waterway immediately downstream of the confluence of two tributaries. The crest level of the small dam should be set below EL. 557.0 m so as to limit the temporally flood inundation area in the park. In addition to the flood control function, those facilities contain a potential to provide the public amenity space and improve the scenery in the urban area. Accordingly, the environmental conditions of pond area would be improved so that the residents will easily and safely access and use the area. A community pond with some stages made through excavation is proposed to use the lower stage for water area and the higher stage for a recreation purpose such as garden, play ground, tennis courts and so on.

The plan and the cross sectional layout of the community pond is shown in Fig. C.5 and the following, respectively.

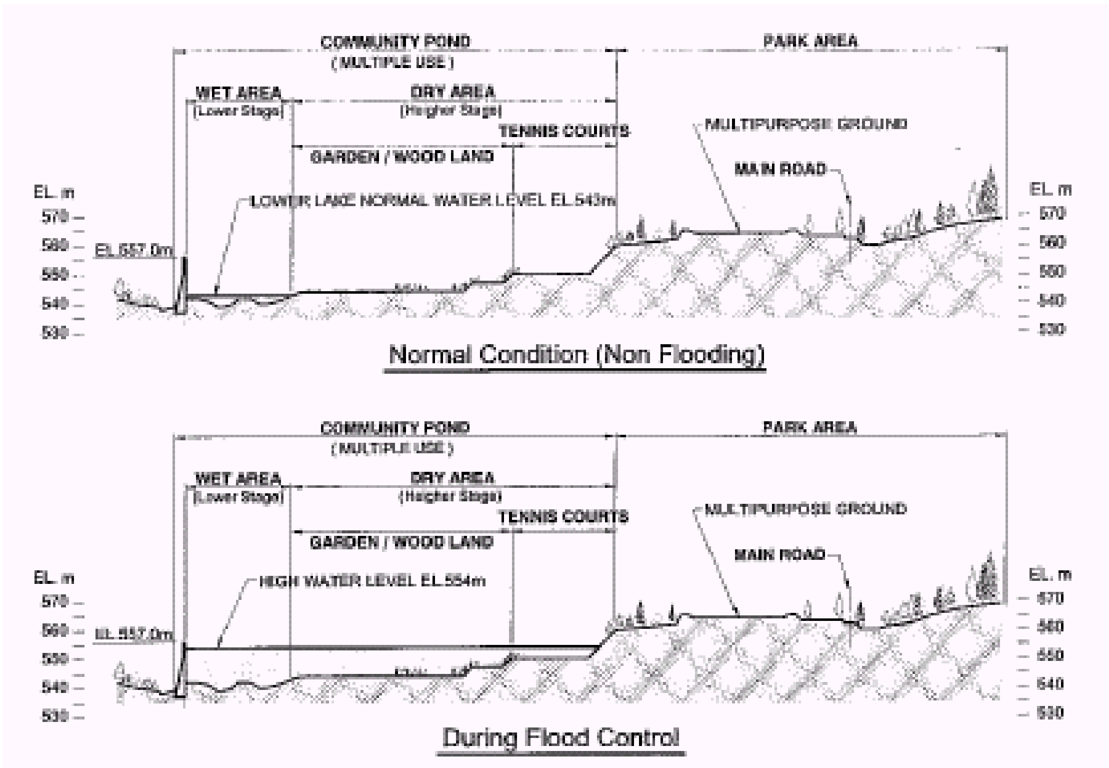


Fig. R C.2 Cross Sectional Layout of Community Pond

### 1.1.4 Flood Mitigation Plan and Reservoir Capacity Allocation

As mentioned in the previous section, the flood control capacity of the community pond is planned as the maximum development so as to limit the temporally flood inundation area in the park. Resulting from this concept, the proposed community pond could have a storage capacity to cut almost all the probable peak runoff discharge of 25-year return period, and reduce about 35% of the peak flood discharge even in case of 100-year return period at site.

These functions could increase the flood safety level of the downstream of Lai Nullah. Calculation results of flood control effect at dam site are given in Fig. C.6 and summarized as below:

Table R C.1 Flood Control Effect of Community Pond at Site

| Return Period | Inflow | | | Regulated Peak Outflow Discharge ($m^3/s$) | Maximum Inundation Level (EL. m) | Maximum Inundation Area ($km^2$) | Peak Discharge Reduction Rate (%) |
|---|---|---|---|---|---|---|---|
| | Diverted Flood Discharge ($m^3/s$) | Flood Discharge ($m^3/s$) | Total ($m^3/s$) | | | | |
| 5-year | 24 | 44 | 68 | 11 | 547.0 | 0.16 | 84 |
| 10-year | 39 | 81 | 120 | 14 | 549.9 | 0.29 | 88 |
| 25-year | 59 | 148 | 207 | 16 | 552.2 | 0.60 | 92 |
| 50-year | 71 | 213 | 284 | 94 | 553.5 | 0.67 | 67 |
| 100-year | 78 | 290 | 368 | 232 | 554.0 | 0.70 | 37 |

Using the topographic map of 1:5,000 newly prepared by the study, pond storage curve of community pond is estimated as shown in Fig. C.7.

### 1.1.5 Design Features of Facilities

Design features of the community pond are summarized as follows:

#### 1) Pond

| | | |
|---|---|---|
| Catchment Area | : | 26.5 $km^2$ (= 16.6 + 9.9 $km^2$) |
| Pond Surface Area | : | 0.64 $km^2$ |
| Maximum Water Surface | : | EL. 555.000 m |
| Surcharge Water Surface | : | EL. 553.000 m |
| Low Water Surface | : | EL. 543.000 m |
| Gross Storage Capacity | : | 2,950,000 $m^3$ |
| Effective Storage Capacity | : | 2,900,000 $m^3$ |
| Dead Storage Capacity | : | 50,000 $m^3$ |

#### 2) Dam Body on Waterway

| | | |
|---|---|---|
| Dam Type | : | Combined Dam |
| Dam Height above Foundation | : | 20.0 m |
| Crest Elevation | : | EL. 557.000 m |
| Foundation Elevation | : | EL. 537.000 m |
| Crest Length | : | 1,550.0 m |

## 1.2 Flood Mitigation Dam

The results of clarification on the possible flood mitigation dams are as described hereinafter:

### 1.2.1 Identification of Potential Dam Sites

A flood mitigation dam has also the function of temporarily storing runoff discharge on the way to the upper reaches of a river and thus flattening the peak runoff discharge. In the same way as the community pond, stored flows are subsequently returned to the downstream river at a reduced rate of flow. It is the core structure for flood regulation in contrast with channel improvement as a core structure for quick disposal of flood discharge.

Generally, the larger the catchment area of a flood mitigation dam, the more effective the flood peak cut. However, potential dam sites for flood mitigation dam are very limited in the study area due to its topographic condition. Almost all parts of the study area are classified into flat land formed on the Potwar plateau, and the mountainous area located at the northern end of the study area is only 15 % of the Lai Nullah basin.

In this study, the potential dam sites were preliminary identified regardless of their catchment area through the review of the previous report, the field reconnaissance and the study on the topographic map newly developed from data of IKONOS. The following six (6) dams are enumerated. Locations of potential dam sites are shown in Fig. C.8.

### 1.2.2 Selection of Optimum Flood Mitigation Dam

Salient features of the six (6) potential dam sites identified in this study are summarized in the following table:

Table R C.2 Features of Potential Dam Sites for Flood Control

| Item | Site-1 | Site-2 | Site-3 | Site-4 | Site-5 | Site-6 |
|---|---|---|---|---|---|---|
| 1. River | Bedarawali Kas | Bedarawali Kas | Bedarawali Kas | Bedarawali Kas | Tenawali Kas | Tenawali Kas |
| 2. Location | Flat Land | Mountainside | Mountainside | Mountainside | Mountainside | Mountainside |
| 3. Geology | Loessic silt, Limestone | Limestone, Sandstone, Shale, Much folded, Many joints, Thick Riverbed | Limestone, Sandstone, Shale, Much folded, Many joints, Thick Riverbed | Limestone, Sandstone, Shale, Much folded, Many joints, Thick Riverbed | Limestone, Sandstone, Shale, Much folded, Many joints, Thick Riverbed | Limestone, Sandstone, Shale, Much folded, Many joints, Thick Riverbed |
| 4. Land Use in Reservoir Area | Belonging to Block E-11, Being illegally developed by Private Developer | Unused Land such as Forest | Unused Land such as Forest | Unused Land such as Forest | Unused Land such as Forest | Unused Land such as Forest |
| 5. Catchment Area ($km^2$) | 19.7 | 1.6 | 2.5 | 3.7 | 1.9 | 4.0 |

In case that each flood mitigation dam has a capacity to cut the probable peak discharge of 100-year return period as much as possible, the design features of each dam and the cost effectiveness are given as below:

Table R C.3 Design Features of Alternative Flood Mitigation Dams

| Item | Site-1 | Site-2 | Site-3 | Site-4 | Site-5 | Site-6 |
|---|---|---|---|---|---|---|
| 1. Required Total Storage Capacity ($m^3$) *1 | 3,040,000 | 250,000 | 390,000 | 560,000 | 290,000 | 610,000 |
| 2. Dam Height (m) | 20.0 | 34.4 | 28.6 | 42.5 | 26.4 | 29.7 |
| 3. Crest Length (m) | 840 | 150 | 155 | 180 | 130 | 230 |
| 4. Area below Maximum Water Level ($km^2$) | 0.80 | 0.04 | 0.07 | 0.07 | 0.06 | 0.11 |
| 5. Embankment Volume ($m^3$) | 300,000 | 358,000 | 217,000 | 521,000 | 164,000 | 378,000 |
| 6. Estimated Peak Cut Discharge (100-year) at Kattarian Bridge ($m^3$/s) | 300 | 26 | 41 | 59 | 30 | 64 |
| | | 220 | | | | |
| 7. Cost (million Rs.) | | | | | | |
| - Construction Cost | 477 | 569 | 344 | 827 | 260 | 601 |
| - Compensation Cost | 1,620 | 4 | 7 | 7 | 6 | 11 |
| - Total Cost | 2,097 | 573 | 351 | 834 | 266 | 612 |
| 8. Cost / Peak Cut Discharge (7./6.) (Rs. / $m^3$/s) *2 | 7,000,000 | 22,000,000 | 8,600,000 | 14,100,000 | 8,900,000 | 9,600,000 |
| | | 12,000,000 | | | | |

*1 The required total storage capacity is estimated on the premise that the flood mitigation dam has a capacity to cut the probable peak discharge of 100-year return period as much as possible at site.

*2 Figures in this column show the cost effectiveness of flood mitigation dam. The smaller figure brings out the higher effectiveness of the flood control function and the greater cost advantage of the facility.

The following matters can be seen in the above table:

(1) Among the identified six (6) potential dam sites, Site-1 located at Block E-11 is greatest advantage in terms of total cost per peak cut discharge in spite of its high compensation cost.

(2) The alternative dams identified at mountainside (Sites-2, 3, 4, 5, 6) have extremely large figures of total cost per peak cut discharge. The reasons are.

- low efficiency of dam reservoir due to their steep riverbed slope,
- low efficiency of flood peak cut discharge due to their small catchment area,
- high cost of foundation treatment due to their weathered and folded foundation.

From the above discussion, Site-1 was selected as the optimum site for flood mitigation dam.

### 1.2.3 Flood Mitigation Plan and Reservoir Capacity Allocation

The proposed flood mitigation dam could have a storage capacity to cut almost all the probable peak runoff discharge of 25-year return period, and reduce about 44% of the peak flood discharge at site even in case of 100-year return period.

These functions could increase the flood safety level of the downstream of Lai Nullah. Calculation results of flood control effect at dam site are given in Fig. C.9 and summarized as below:

Table R C.4 Flood Control Effect of Flood Mitigation Dam at Site

| Return Period | Inflow ($m^3/s$) | Regulated Peak Outflow Discharge ($m^3/s$) | Reservoir Surface Level (EL. m) | Peak Discharge Reduction Rate (%) |
|---|---|---|---|---|
| 5-year | 45 | 4 | 567.1 | 91 |
| 10-year | 86 | 5 | 568.5 | 94 |
| 25-year | 162 | 6 | 570.8 | 96 |
| 50-year | 236 | 65 | 572.2 | 72 |
| 100-year | 325 | 183 | 572.8 | 44 |

Using the topographic map of 1:5,000 newly prepared by the study, reservoir storage curve of the flood mitigation dam is estimated as shown in Fig. C.10.

### 1.2.4 Design Features of Flood Mitigation Dam

The plan is shown in Fig. C.11. Design features of the flood mitigation dam are summarized as follows:

#### 1) Reservoir

| | | |
|---|---|---|
| Catchment Area | : | 19.7 $km^2$ |
| Reservoir Surface Area | : | 0.62 $km^2$ |
| Maximum Water Surface | : | EL. 574.000 m |
| Surcharge Water Surface | : | EL. 571.600 m |
| Low Water Surface | : | EL. 565.300 m |
| Gross Storage Capacity | : | 3,040,000 $m^3$ |
| Effective Storage Capacity | : | 2,640,000 $m^3$ |
| Dead Storage Capacity | : | 400,000 $m^3$ |

#### 2) Dam Body on Waterway

| | | |
|---|---|---|
| Dam Type | : | Fill Dam |
| Dam Height above Foundation | : | 20.0 m |
| Crest Elevation | : | EL. 576.000 m |
| Foundation Elevation | : | EL. 556.000 m |
| Crest Length | : | 840.0 m |
| Crest Width | : | 5.0 m |
| Embankment Slope | : | Upstream 1:3.5, Downstream 1:3.0 |

### 1.3 Flood Diversion Channel

The results of clarification on the possible flood diversion channel are as described hereinafter:

#### 1.3.1 Preliminary Screening of Potential Diversion Channel Routes

As the final solution of the flood problem, diversion channel to adjacent rivers has been studied somewhere upstream of Rawalpindi city so that no flood passes through the city area. The routes of the diversion channel examined in the previous study and in this study are shown in the following figure and table.

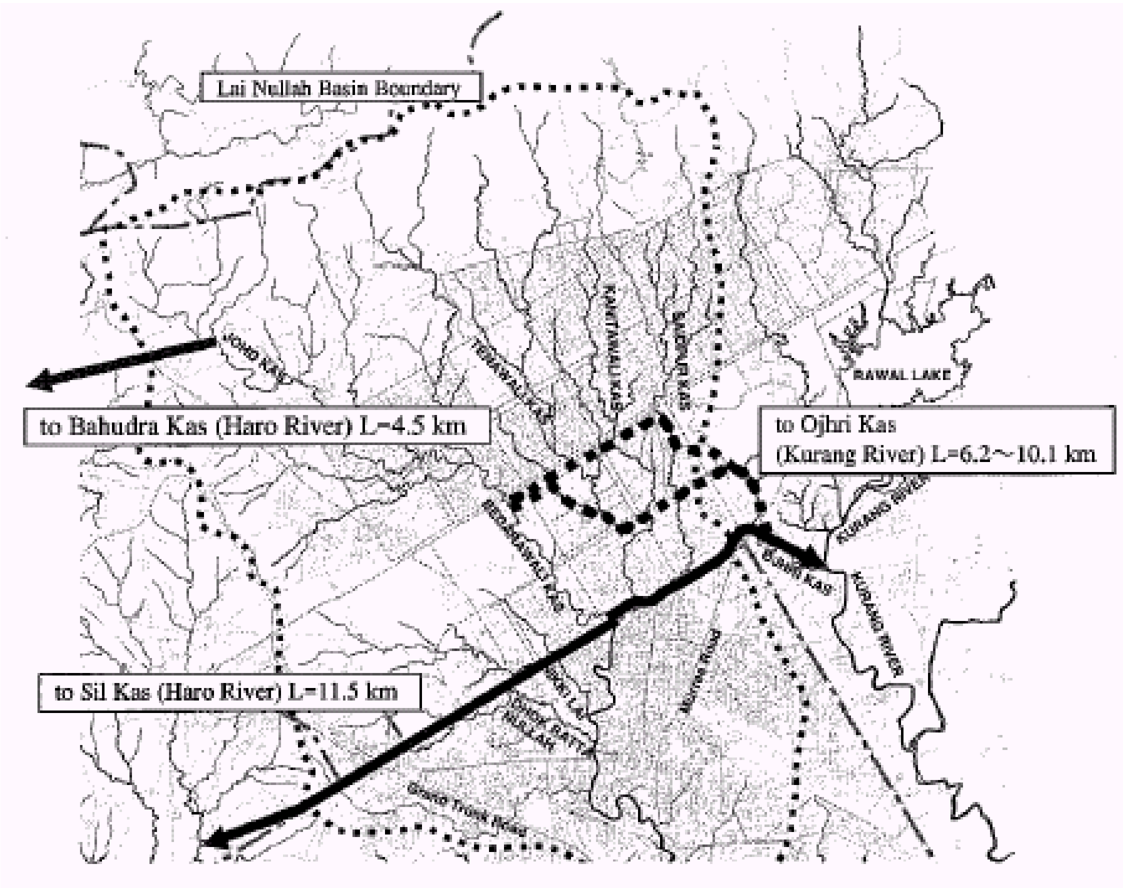


Fig. R C.3 Potential Routes of Flood Diversion Channel

Table R C.5 Potential Routes of Flood Diversion Channel

| Routes of Diversion Channel | Diverted River/Tributary | Catchment Area ($km^2$) | Length of Channel (km) |
|---|---|---|---|
| to Bahudra Kas of Haro River | Johd Kas (Bedarawali Kas) | 12 | 4.5 |
| to Sil Kas of Haro River | Bedarawali Kas, Nikki Lai Dhok Ratta Nullah | 103 | 11.5 |
| to Ojhri Kas of Kurang River | Bedarawali Kas, Tenawali Kas Kanitawali Kas, Saidpur Kas | 122 to 144 | 6.2 to 10.1 |

The diversion channel to the Bahudra Kas of the Haro River does not have any difficulties of land acquisition. However, its possible catchment area of tributary to be diverted is limited to only 12 $km^2$, equivalent to about 5 % of the Lai Nullah basin, and therefore, this diversion could not provide adequate relief.

The diversion channel to the Sil Kas of the Haro River planed to cut across hill area. Difference of land level among tributaries to be diverted and top of hill area is not less than 60 m. Thus, the extremely large excavation volume is required and, therefore this route is not practicable.

After exclusion of inappropriate routes, the diversion channel to the Ojhri Kas of the Kurang River remains as alternative routes to be studied.

### 1.3.2 Features of Alternative Routes to divert flood into Kurang River

Three (3) alternative routes to divert flow into Kurang River can be considered. They divert the flow of the four (4) main tributaries, namely, Bedarawali Kas Tenawali Kas, Kanitawali Kas and Saidpur Kas, and run through the urbanized area of Islamabad between the existing roads (conservation area as a greenbelt) and finally outfall into Kurang River. Plan and longitudinal profiles of alternatives are shown in Figs. C.12 and C.13. Salient features of these alternatives are summarized as follows:

Table R C.6 Salient Features of Alternative Routes

| No. | Route of Diversion Channel | Catchment Area ($km^2$) | Riverbed Level (EL. m) | | Possible Riverbed Slope | Channel Length (m) |
|---|---|---|---|---|---|---|
| | | | Bedarawali Kas | Kurang River | | |
| Route-1 | along Kashimir Highway | 122 | 515.0 | 488.0 | 1/700 | 10,155 |
| Route-2 | along Khayaban-E-Johar Road (one block south from Route-1) | 129 | 515.0 | | 1/700 | 9,726 |
| Route-3 | along Khyaban-E-Siryed Road (called I-J Principal Road) | 144 | 495.0 | | 1/1000 | 6,233 |

### 1.3.3 Selection of Optimum Route

The allowable maximum capacity of flood diversion channel is estimated at about 1,700 $m^3/s$ taking the following factors into account: (a) the allowable limit of the right-of-way for the diversion channel, (b) the possible bed of the diversion channel and (c) the required improvement works of Kurang River as the outlet of the diversion channel[1]. The study on the optimum route of the diversion channel is carried out on the assumption that the capacity of

[1] CDA commented in the Steering Committee Meeting on the Draft Final Report that the right-of-way for the route-2 of the diversion channel should be restricted to be a certain width. Due to the comment, the possible maximum diversion discharge for the alternative route-2 may fall below 1,700$m^3/s$. After detailed discussions, it is finally agreed by the Steering Committee that this matter would be clarified in the succeeding Feasibility Study (refer to item 2 in the Minutes of Steering committee Meeting on the Draft Final Report as attached to this Main Report).

flood diversion is fixed at 1,480 $m^3/s$, which corresponds to the design discharge of the diversion channel, if the proposed community pond (assumed as the strongest candidate of the priority project component) is constructed in the upper reaches. The required cross sectional area of each alternative is given in Fig. C.14. Measure work quantities and compensation works are summarized as follows:

Table R C.7 Measure Work Quantities and Compensation Works for Alternative Diversion Route

Measure Work Quantities

| No. | Excavation ($m^3$) | Dike Embankment ($m^3$) | Side Slope Protection ($m^2$) | Sodding ($m^2$) | Bridge (bridges) |
|---|---|---|---|---|---|
| Route-1 | 7,900,000 | 70,000 | 158,000 | 295,000 | 12 |
| Route-2 | 4,000,000 | 131,000 | 164,000 | 167,000 | 20 |
| Route-3 | 5,000,000 | 16,000 | 106,000 | 153,000 | 16 |

Compensation Works

| No. | Land Acquisition ($m^2$) | | | House Evacuation (houses) | | |
|---|---|---|---|---|---|---|
| | Residential Area | Others | Total | in I-8, I-9 | along Ojhri Kas | Total |
| Route-1 | 5,700 | 312,000 | 317,700 | 0 | 19 | 19 |
| Route-2 | 6,000 | 342,000 | 348,000 | 0 | 20 | 20 |
| Route-3 | 41,900 | 176,000 | 217,900 | 76 | 13 | 89 |

From the above studies, cost of each alternative route is estimated as below:

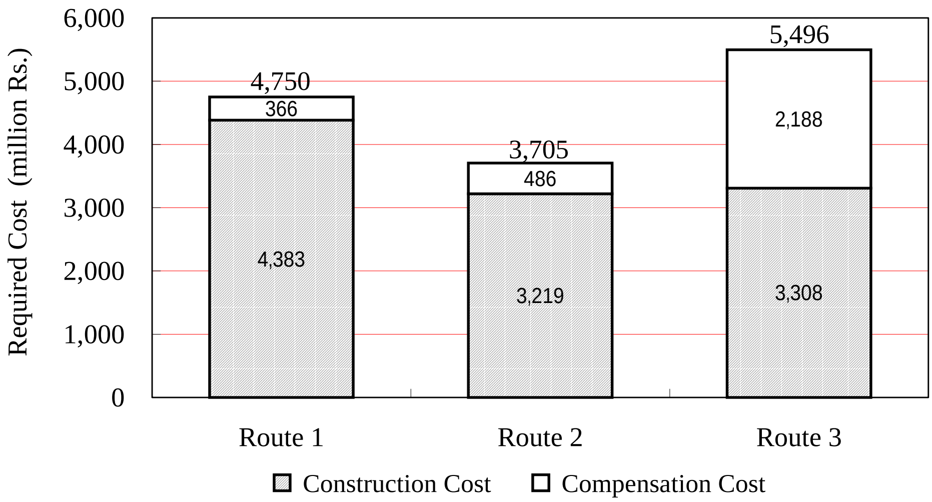


Fig. R C.4 Cost Comparison of Alternative Routes

It is concluded that the route-2 is the most suitable alternative for diversion channel to divert flood into Kurang River due to the following reasons:

(1) The route-2 is the most economical alternative, when their construction cost and compensation cost are contrasted.

(2) It is deemed to be difficult to implement construction of route-3 diversion channel due to difficulties in evacuating many permanent houses located in Blocks I-8 and I-9.

(3) In case of route-2, no house evacuation in Blocks I-8 and I-9 is necessary.

### 1.3.4 Design Features of Flood Diversion Channel

Design features of the flood diversion channel (Route-2) are summarized as follows:

| | | |
|---|---|---|
| Catchment Area | : | 129 $km^2$ |
| Channel Bed Slope | : | 1/700 |
| Channel Length | : | 9,726 m in total |
| | : | 2,450 m (Bedarawali Kas - Tenawali Kas) |
| | : | 2,150 m (Tenawali Kas - Saidpur Kas) |
| | : | 5,126 m (Saidpur Kas - Kurang River) |

### 1.3.5 Necessary Treatment for Kurang River

The proposed flood diversion channel flows into Kurang River through its tributary named Ojhri Kas and finally pours into Soan River. Between these two (2) outlet rivers, Soan River has unquestionably the adequate channel flow capacity to accommodate the flood discharge from Kurang River as well as Lai Nullah. On the other hand, there are the bottleneck stretches along Kurang River, which cause the frequent flood overflow. In order to safely divert the proposed flood discharge into Kurang River, the necessary treatment works for the River was preliminarily evaluated taking its present channel flow capacity and the flood runoff discharge from the river basin into account.

#### 1) Flood Discharge of Kurang River

There exists Rawal Dam on Kurang River about 5.6 km upstream from the outlet point of the flood diversion (i.e., at the confluence of Ojhri Kas, the tributary of Kurang River). The dam reservoir is used as the major source for water supply to Rawalpindi, but at the same time, it has a certain effect on the flood mitigation for the downstream of Kurang River. That is, the water stage of the dam reservoir drops to EL. 531 m in the early of July from the normal water level of EL 532 m, and then it gradually rises

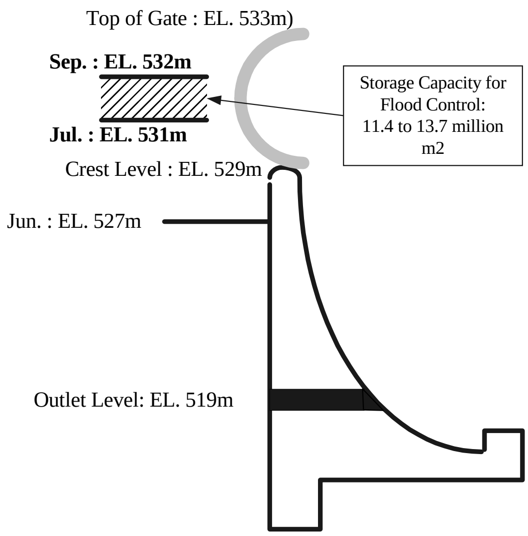


Fig. R C.5 Water Stage of Rawal Dam Reservoir finally recovering to the normal level in the end of September. The drop of water stage in the early of July could create a stage capacity of 13.7 million $m^3$ for flood control (refer to Fig. R C.5). This flood control capacity could reduce the peak flood runoff discharge from the upper reaches of the dam and delay the time of occurrence of the peak discharge. In the flood of July 2001, the dam reservoir released its peak discharge of only 220 $m^3/s$ after the flood of downstream is subsided.

The above flood control capacity is, however, not always expected due to the gradual raise of the reservoir water stage as stated above. Depending on the timing of flood occurrence, the dam may possibly release the substantial discharge. According to the record of the dam outflow discharge, the annual maximum dam outflow discharge fluctuates year-by-year, and the largest value of 1,300 $m^3/s$ was recorded in 1988 as shown in Fig. R C.6.

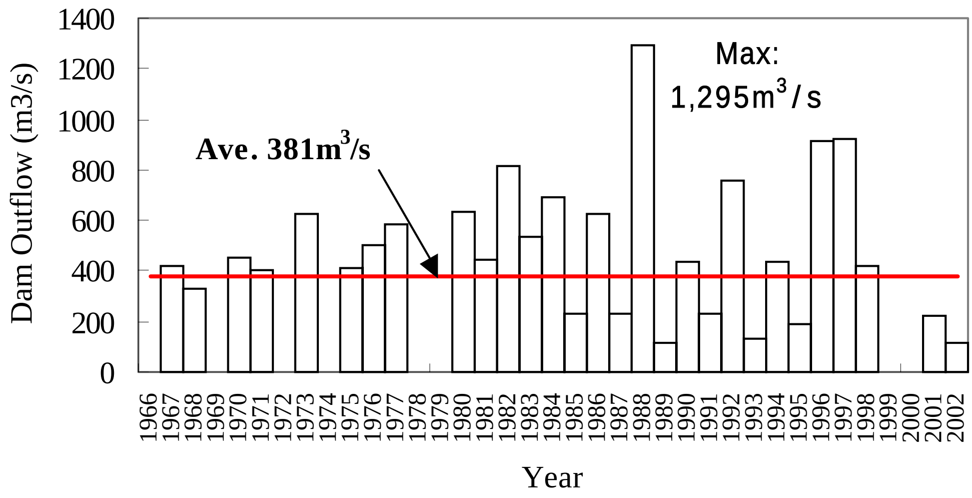


Fig. R C.6 Annual Maximum Outflow Discharge from Rawal Dam

Thus, the peak dam outflow discharge is changeable depending on the complex factors of dam water stage, timing of flood occurrence and volume of the runoff discharge from the upper reaches of the dam reservoir.

Due to the above complex factors, it is virtually difficult to determine the designed dam outflow discharge through hydrological simulation. In this Study, however, the maximum dam outflow of about 1,300 $m^3/s$ recorded in 1988 is provisionally assumed as the design discharge released from Rawal dam. The following items are further assumed, and it is concluded that the maximum peak discharge to be accommodated by Kurang River would range from 2,530 $m^3/s$ to 3,240 $m^3/s$ at outlet point of the diversion channel (i.e., the confluence point with Ojhri Kas):

(a) The peak flood runoff discharge of 100-year return period from the catchment area of Ojhri Kas is estimated at 310 $m^3/s$ through the hydrological simulation.

(b) The maximum discharge from the diversion channel would be about 1,630 $m^3/s$ in the flood of 100-year return period assuming that any flood storage structures (i.e., the community pond in Fatima Jinnah Park and/or flood mitigation dam in Block E-11) are not constructed in the catchment area of the diversion channel.

(c) It is assumed as the worst case, that the peak discharge from the diversion discharge could coincide with the peak discharge from Rawal dam and the catchment area of Ojhri Kas. In this case, the peak discharge to be accommodated by Kurang River is estimated to at about 3,240 $m^3/s$ as the total of 1,630 $m^3/s$ from the diversion channel, 1,300 $m^3/s$ from Rawal Dam and 310 $m^3/s$ the catchment area of Ojhri Kas.

**2) Existing Channel Flow Capacity of Kurang River**

The upstream channel of about 4.4 km in length from the outlet point of the proposed diversion channel to the confluence of Gumreh Kas (tributary of Kurang River) has U-shape cross-sections with the channel depth of only about 2 m, although it has the rather large channel width of more or less 100 m. Moreover, the upstream channel has the very gentle channel bed slope of about 1/1,500. Due to these characteristics, the channel flow capacity of the upper reaches is limited to about 200 $m^3/s$, which is far smaller than the aforesaid expected maximum flow discharge of 3,240 $m^3/s$ for Kurang River as listed below.

Table R C.8 Hydraulic Channel Dimensions and Channel Flow Capacity of Kurang River

| Name of Point | Distance from Rawal Dam Site (km) | Channel Bed Slope | Maximum Depth (m) | Maximum Width (m) | Hydraulic Radius (R) (m) | Maximum Discharge ($m^3/sec$) |
|---|---|---|---|---|---|---|
| Soan Village | 5.6 | 1/750 | 5.8 | 130 | 3.0 | 960 |
| Shakrial Village | 7.3 | 1/1500 | 2.1 | 110 | 1.3 | 140 |
| Khanna Bridge | 10.0 | 1/140 | 2.9 | 96 | 1.8 | 730 |
| Karal Village | 12.5 | 1/140 | 11.0 | 75 | 6.8 | 5,300 |

Source : Results of river channel survey by Small Dam Organization in 2001
Note : The villages of Soan, Shakrial and Khanna are located upstream from the confluence of Gumreh Kas, while Karal Village is downstream from the confluence of Gumreh Kas.

In contrast with the upstream, the downstream of Kurang River with a length of about 16 km between the confluences of Gumreh Kas (the tributary of Kurang River) and Soan River has the steep cliff at both of the left and right banks, and the steep channel bed slope of about 1/140. According to the results of the field reconnaissance and the uniform calculation based on the channel survey by the Small Dam Organization, the channel flow capacity of the downstream stretch is evaluated to be more than 5,300 $m^3/s$ as listed above, which could adequately accommodate the above peak discharge of 3,240 $m^3$/s, even if the flood runoff discharge from the catchment area of Gumreh Kas is added.

It is, however, herein noted that the land development for the new residential area is now in progress at and around the confluence of Kurang River and Soan River. Through the land development, the river channel of Kurang along the residential has been filled-up, and the new short-cut channel has been constructed. According to the site investigation, the short-cut channel deems to have far lower channel flow capacity than its upstream channel, and the residential area itself becomes the great hindrance to discharge the flood flow of Kurang River into Soan River. In order to offset such unfavorable conditions, it is indispensable to immediately suspend the on-going land development and restore the channel flow capacity as in the past, regardless of construction of the flood diversion channel from Lai Nullah to Kurang River.

### 3) The Areas to be protected against Flood Overflow of Kurang River

There exist three (3) settlement areas, namely, Soan, Shikrial and Khanna Dak, along the upstream channel of Kurang River. These villages currently suffer the habitual flood inundation by the overflow from Kurang River. Should the proposed flood diversion channel be constructed, the flood flow discharge of Kurang River definitely increases and accretes the present flood damage to the villages in particular (refer to Fig. R C.7).

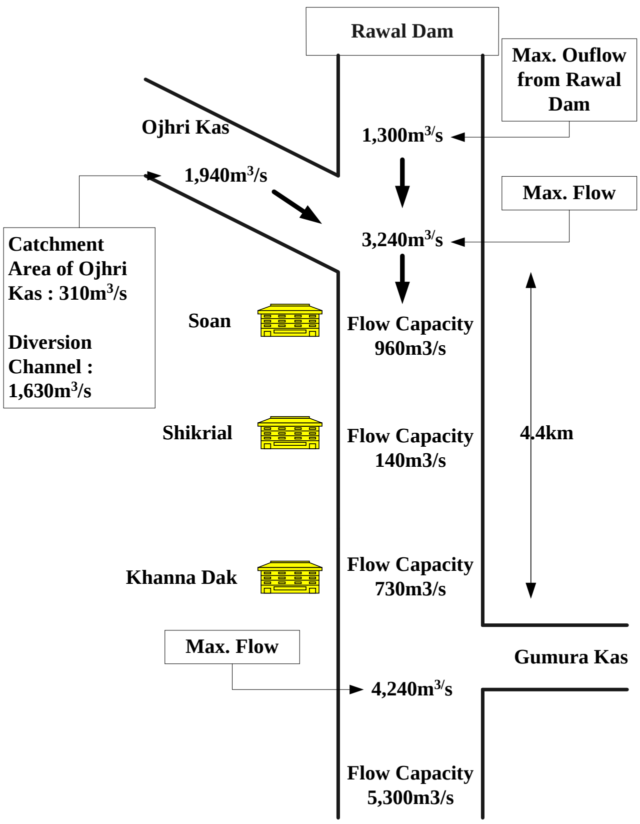


Fig. R C.7 Channel Flow Capacity and Design Flow Discharge of Kurang River

Hence, the certain flood protection for the villages would become an indispensable precondition for selection of the flood diversion option. Nevertheless, apart from the settlement area of the villages, the substantial part of the rive-side along the upstream of Kurang is remained as the natural unused land and/or agricultural land. Accordingly, the major target of the objective flood mitigation could be limited to the settlement areas of the three (3) villages.

### 4) Flood Mitigation Measures Required to Kurang River

In order to offset the increment of flood damage potential of Kurang River inflicted by construction of the proposed flood diversion channel, establishment of the river reserve area and construction of the ring dike is proposed as shown in Fig. C.15. The required work volumes for these proposed flood mitigation works are preliminarily estimated as listed in Table R C.9. These are, however, subject to revision based on the further detailed topographic survey and hydrological analysis on the flood runoff discharge and the flood inundation.

Table R C.9 Required Work Volume for Proposed Ring Dike

| Work Item | Work Volume |
|---|---|
| 1. Length Ring Dike | |
| 1.1 Right Dike for Soan Village | 1,300 m |
| 1.2 Right Dike for Shikrial Village | 1,570 m |
| 1.3 Right Dike for Khanna Dak Village | 1,430 m |
| 1.4 Left Dike for other dotted settlement areas | 2,200 m |
| Total | 6,500 m |
| 2. Extent of Land Acquisition for Establishment of River Reserve Area and Construction of Ring Dike | 334,000 $m^2$ |
| 3. Number of Necessary House Evacuation | 220 houses |

Details of the proposed river reserve area and ring dike are as described in the following items (a) to (c):

(a) The area long the section of 10,930m from Rawal Dam to the confluence of Gumura Kas should be delineated and gazetted to be the river reserve area as the buffer against the flood overflow and the right-of-way for the future river channel improvement works. Any unfavorable land development within the river reserve area should be prohibited.

The CDA has already declared the left and right bank of 1,000 feet in width each from the center of the river course as the river reserve area. However, the width of 2,000 feet (about 600 m) in total covers the substantial part of the existing settlement area, and at the same time, it deems to be too spacious as compared with the potential extent of the flood inundation and the necessary extent for the future river improvement. From these viewpoints, the extent of the river reserve area is provisionally proposed at 200m in width from the center of the river cause (refer to Fig. C.15).

(b) A certain structural flood mitigation measures for the aforesaid three (3) villages namely Soan, Shikrial and Khanna Dak along Kurang River would be required to relive the villages from the adverse effect inflicted by construction of the proposed flood diversion channel. The villages are, however, rather sparsely dispersed along Kurang River, and therefore, the river channel improvement for the entire river stretch is not required. Instead, the ring dike is proposed to besiege the villages and prevent them from flood overflow of Kurang River.

(c) Execution of the above river reserve area and the ring dike would require evacuation of about 220 houses. Such substantial number of house evacuation may create a social conflict and therefore would be addressed as the important issue for achievement of the proposed flood mitigation works for Kurang River. Nevertheless, the houses as the objectives of evacuation are even now exposed to the frequent flood damage and, any measure other than evacuation is not practical to get rid of such unfavorable conditions.

Moreover, urban population of Rawalpindi is now being spilled over the possible flood inundation area particularly along right bank of Kurang River. Should the area along Kurang River be left behind without clearance of houses within the extent of the proposed river reserve area, the riverside along Kurang River would be finally saturated with the house and buildings like the current situation of Lai Nullah and remarkably increase the flood damage potential. In order to avoid such unfavorable conditions, the house evacuation would be indispensable even regardless to construction of the proposed flood diversion channel.

## 2. STUDY ON SELECTED FLOOD MITIGATION STRUCTURES

The optimum flood mitigation plan was finally determined through comparison of the alternative schemes based on the necessary working period, the project cost, the compensation works and other relevant social/natural environmental conditions (refer to Sector D).

The community pond at Fatima Jinnah Park and the flood diversion channel as the principal components of the schemes were selected for the optimum flood mitigation scheme. Distribution of design flood discharge is shown as below:

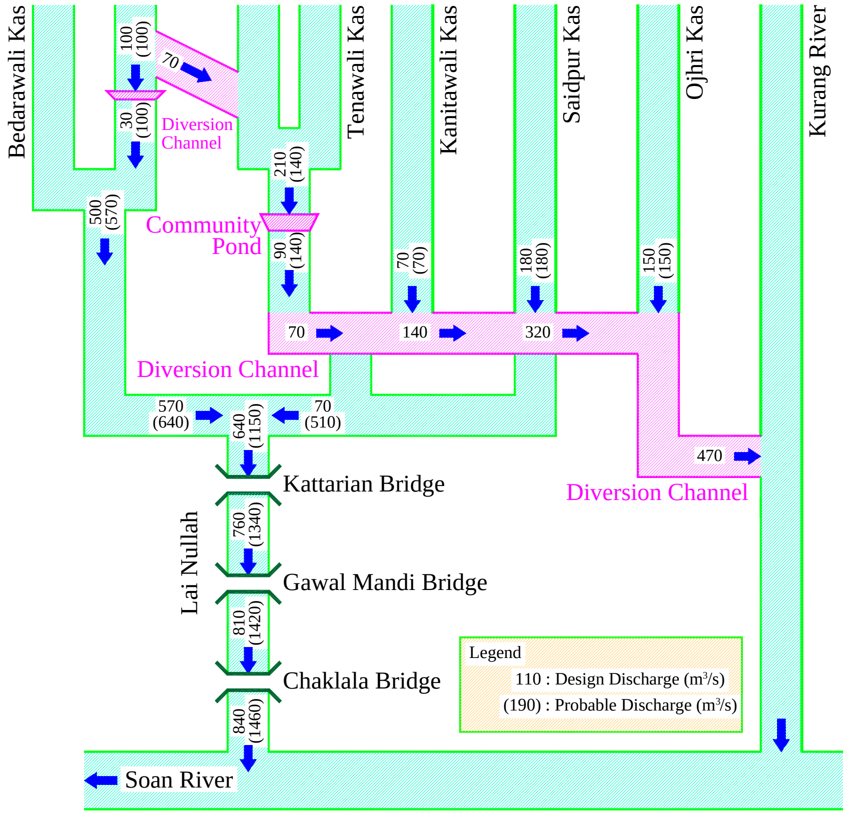


Fig. R C.8 Distribution of Design Flood Discharge (25-year Return Period)

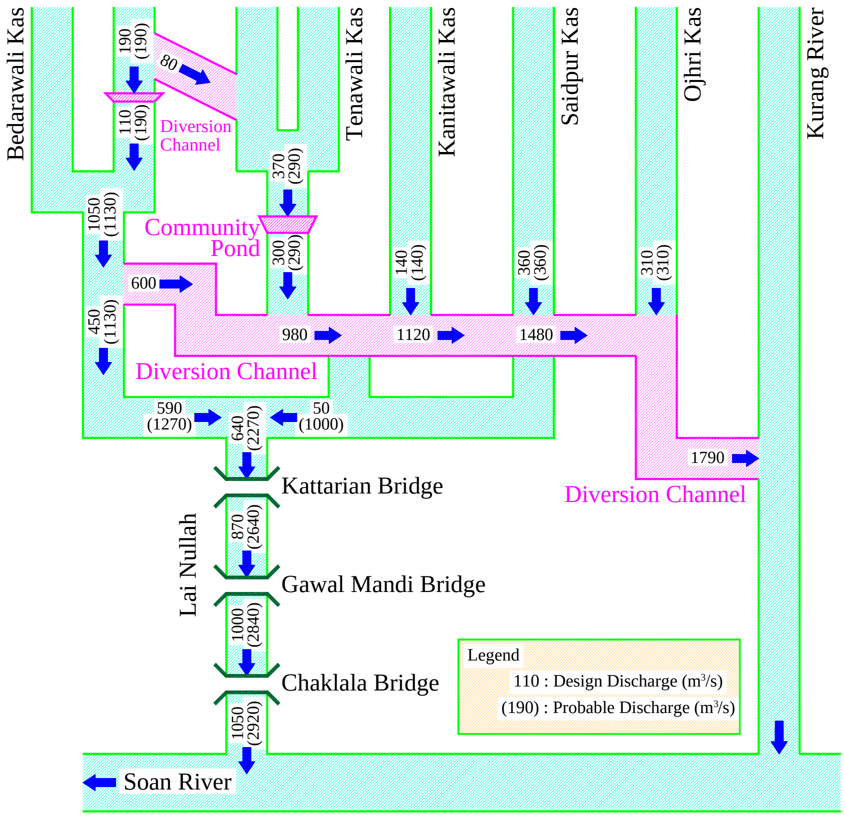


Fig. R C.9 Distribution of Design Flood Discharge (100-year Return Period)

## 2.1 Preliminary Design of Facilities

### 2.1.1 Community Pond

#### 1) Detention Dam and Appurtenant Structures

##### a) Embankment Dam Body

6 m of the crest width is adopted considering proper construction procedure and width of crest road for maintenance. The upstream slope and downstream slope are designed at 1.0 vertical to 3.5 horizontal and 1.0 vertical to 3.0 horizontal, respectively to secure the stability of both slopes.

##### b) Spillway

Spillway is provided to release surplus or flood water, which cannot be contained in the allotted storage capacity of the pond. It should have sufficient capacity to accommodate the flood peak discharge equivalent to 120 % of 200-year return period for an embankment dam.

*Flood Peak Discharge* = *466 $m^3/s$ (200-year return period, 26.5 $km^2$) x 120 %*
= *560 $m^3/s$*

Spillway is designed as concrete gravity type on the existing river, in which over flow crests are equipped. The crest level is set at the Surcharge Water Surface EL. 553.0 m. 2 m in over flow depth and 100 m in width are adopted.

*Capacity of Over Flow Crests* = $C \times B \times H^{3/2} = 2.0 \times 100.0 \times 2.0^{3/2}$
= *565 $m^3/s$ > 560 $m^3/s$ OK*

*C: Coefficient of Over Flow Discharge = 2.0*
*B: Over Flow Width = 100.0 m*
*H: Over Flow Depth = 2.0 m*

##### c) Orifice for Flood Mitigation

Orifice should have a function to regulate a 25-year probable flood with peak discharge of 210 $m^3/s$ through the pond. Two (2) orifices (1 m in height, 1 m in width) are equipped in the concrete gravity type spillway. A steel screen is also built at slightly far from orifices in order to prevent the orifices from clogging by drifting rubbish. Slide gates for orifices also should be installed to facilitate easy and sustainable maintenance work. The level of the orifice is set at EL. 543.0 m (Low Water Level). The flood control capacity between the Low Water Level and the Surcharge Water Level is 2,900,000 $m^3$ including 20 % allowance.

**2) Diversion Channel to Community Pond**

By the fixed weir planned on Bedarawali Kas, floodwater from the upper reach of Bedarawali Kas is dammed up and excessive flood discharge is diverted to the community pond through the diversion channel. The diverted flow discharge is regulated by the orifices equipped in the diversion weir planned at the beginning of the diversion channel. Height of the fixed weir and the diversion weir is 2.5 m and 5.2 m, respectively. Length of the diversion channel is 1,340 m (refer to Fig. C.16). Diverted flood discharge is calculated in the following table:

Table R C.10 Diverted Flood Discharge from Bedarawali Kas

| Return Period | Flood Discharge from Upper Reach of Bedarawali Kas ($m^3/s$) | Flood Discharge to Downstream of Bedarawali Kas ($m^3/s$) | Flood Discharge to Community Pond ($m^3/s$) |
|---|---|---|---|
| 5-year | 31 | 7 | 24 |
| 10-year | 55 | 16 | 39 |
| 25-year | 97 | 38 | 59 |
| 50-year | 137 | 77 | 71 |
| 100-year | 183 | 105 | 78 |
| 200-year | 237 | 151 | 86 |

**3) Facilities for Water Quality Control**

The river water in and around the study area is heavily polluted, because the river and tributaries are receiving large volume of polluted wastewater generated in the urbanized area. The pollution level is very high even around the Fatima Jinnah Park located in the upper reaches of the Lai Nullah. Under these circumstances, without any proper countermeasures to treat collected wastewater, the water filled in the proposed wet pond might emit offensive odor to the neighborhoods, especially in dry-weather. In order to avoid this unfortunate prediction, the following facilities are proposed (refer to Fig. C.17).

**a) Inlet Gate and Bypass Pipe for Heavily Polluted Wastewater**

Heavily polluted wastewater from the left tributary of Tenawali Kas should bypass the community pond during dry season, because runoff water from Margalla Hill, which is diluted wastewater with, might be dried up in dry-weather.

Inlet gate is proposed to be constructed just upstream of the existing box culverts near the eastern entrance of the park. The bypass pipe is installed at the bare land between the fence of the park and the existing road. The bypassed wastewater is then discharged to Tenawali Kas downstream of the community pond.

**b) Intake Gate for Less Contaminated Water from Bedarawali Kas**

According to our site investigation, water quality of the tributary of Bedarawali Kas is comparatively better than the tributary of Tenawali Kas. In order to lead to a change in water quality of the Community Pond, intake gate is proposed to be equipped in the diversion weir so as to take less contaminated water. The water from Bedarawali Kas flows into the community pond through the diversion channel.

**c) Oxidation Pond**

Oxidation Ponds are known as stabilization ponds or lagoons, which are used for simple secondary treatment of sewage effluents. Oxidation pond is proposed on each tributary to treat the polluted wastewater, which flows into the community pond.

Within an oxidation pond heterotrophic bacteria degrade organic matter in the polluted water, which results in production of cellular material and minerals. The production of these in the oxidation pond supports the growth of algae, which allows further decomposition of the organic matter by producing oxygen.

**d) Orifice for Drawdown**

As an applicable structural measure to prevent the environmental deterioration in the community pond, an orifice equipped in the concrete gravity type spillway is proposed. It is able to drawdown the water level and finally dries up the wet pond.

**4) Check Dam for Sedimentation and Garbage**

Margalla Hill, which is the greater part of the catchment area of the community pond, is formed with limestone, calcareous conglomerates and red shale. These rocks formed in the much old era are hard and massive. Most of surface deposits had been flashed out and outcrops are observed everywhere. From the above geological condition, the sediment yield in the catchment area is very small and most of sedimentation is estimated to be composed of sand and gravel.

To prevent that these sand and gravel flow into the community pond, check dam for sedimentation with 1.5 m in height, which is constructed of wet stone masonry, is proposed on each tributary (refer to Fig. C.17). The check dam, which is a small barrier placed across a stream, can reduce the effective slope and create small pool in the stream. Reduced slope can reduce the velocity of storm water flows and promote sedimentation behind the dam. The check dam will also be effective to stop the inflow of garbage.

### 5) Area and Facilities for Multiple Use of Community Pond

In addition to the flood detention function, those facilities contain a potential to provide the public amenity space and improve the scenery in the urban area. The community pond with some stages made through excavation is proposed to use each stage for a recreation purpose such as garden, play ground, tennis courts taking inundation frequency and duration into account (refer to Figs. C.18 and C.19).

Table R C.11 Inundation Frequency and Duration

| Return Period | Maximum Inundation Level (EL. m) | Maximum Inundation Area ($km^2$) | Duration of Inundation (hour) | | |
|---|---|---|---|---|---|
| | | | Lower Stage EL. 544.0 m | Middle Stage EL. 547.0 m | Upper Stage EL. 550.0 m |
| 5-year | 547.0 | 0.16 | 21 | - | - |
| 10-year | 549.9 | 0.29 | 36 | 20 | - |
| 25-year | 552.2 | 0.60 | 61 | 46 | 29 |
| 100-year | 554.0 | 0.70 | 73 | 57 | 40 |

In this study, the following facilities are proposed at each stage:

(a) Lower Stage (EL. 544.0 m): Flower Garden, Wood Land, Open Space

(b) Middle Stage (EL. 547.0 m): Flower Garden, Wood Land, Open Space

(c) Upper Stage (EL. 550.0 m): Tennis Courts, Multipurpose Ground, Flower Garden

During the rainy season (July to September), personnel visited to the park is not allowed to enter the area below EL. 547.0 m including the lower and middle stages, because this area is designed to be inundated by flood less than 10-year return period probability. The signboard that says “RESTRICTED AREA, Authorized Personnel Only beyond This Point“ and fence around the restricted area should be installed.

Even during the rainy season, the upper stage at EL. 550.0 m can be used by personnel, provided that level of risk should be identified and informed to personnel. The signboard that says “WARNING, Water subject to Sudden Rise and Turbulence” or “WARNING, Place subject to Sudden Flooding” should be installed.

### 6) Design Features of Facilities

Design features of the necessary facilities are summarized as follows (refer to Fig. C.17):

#### a) Pond

| | | |
|---|---|---|
| Catchment Area | : | 26.5 $km^2$ (= 16.6 + 9.9 $km^2$) |
| Pond Surface Area | : | 0.64 $km^2$ |
| Maximum Water Surface | : | EL. 555.000 m |
| Surcharge Water Surface | : | EL. 553.000 m |
| Low Water Surface | : | EL. 543.000 m |
| Gross Storage Capacity | : | 2,950,000 $m^3$ |
| Effective Storage Capacity | : | 2,900,000 $m^3$ |
| Dead Storage Capacity | : | 50,000 $m^3$ |

#### b) Dam Body on Waterway

| | | | | |
|---|---|---|---|---|
| Dam Type | : | Combined Dam | | |
| Dam Height above Foundation | : | 20.0 m | | |
| Crest Elevation | : | EL. 557.000 m | | |
| Foundation Elevation | : | EL. 537.000 m | | |
| Crest Length | : | 1,550.0 m | Embankment Type | L = 1,260 m |
| | | | Concrete Type | L = 290 m |
| Crest Width | : | 6.0 m | | |
| Concrete Gravity Portion | : | Upstream Vertical, Downstream 1:0.8 | | |
| Embankment (Homogeneous) Portion | : | Upstream 1:3.5, Downstream 1:3.0 | | |

#### c) Spillway and Outlet Facilities for Dam

| | | |
|---|---|---|
| Design Flood (Inflow Peak Discharge) | | |
| Emergency Spillway Design Flood | : | 560 $m^3/s$ (200-year probability x 120 %) |
| Flood Control Capacity | : | 210 $m^3/s$ (25-year Probability) |
| Overflow Crest | : | Crest EL. 553.000 m, 100 m in length |
| Orifice for Flood Mitigation | : | H 1.0 m x W 1.0 m x 2, EL. 543.0 m |

#### d) Diversion Channel to Community Pond

| | | |
|---|---|---|
| Fixed Weir on Tributary | : | 2.5 m in height, 37 m in length |
| | : | Overflow Crest L = 16 m |
| | : | Orifice H 1.0 m x W 1.0 m x 2 |
| Diversion Weir with Orifice | : | 5.2 m in height, 20 m in length |
| | : | Orifice H 1.0 m x W 1.0 m x 1 |
| | : | Orifice H 1.5 m x W 1.5 m x 7 |
| Diversion Channel (Wet Stone Masonry) | : | 8 m in width, 1,340 m in length |
| | : | Water Depth D = 2.4 m |

| | | |
|---|---|---|
| | : | Channel Bed Slope I = 1/200 |

**e) Facilities for Water Quality Control**

| | | |
|---|---|---|
| Inlet Gate for Bypass Pipe | : | H 1.0m x W 1.0 m x 1 |
| Bypass Pipe | : | D 0.6 m x L 1,600 m |
| Intake Gate at Bedarawali Kas | : | H 1.0 m x W 1.0 m x 1 |
| Oxidation Pond | : | A = 5,000 $m^2$ x 2 |
| Orifice for Drawdown (in Spillway) | : | H 1.0 m x W 1.0 m x 1, EL. 540.0 m |

**f) Check Dam for Sedimentation and Garbage**

| | | |
|---|---|---|
| Weir (Wet Stone Masonry) | : | H 1.5m x L 20 m x 1 |
| Weir (Wet Stone Masonry) | : | H 1.5m x L 30 m x 1 |

**g) Facilities for Multiple Use of Community Pond**

Public Facilities (Road, Bridge, Car Parking, Backfilling, etc.)
Sports and Recreation facilities (Multipurpose Ground, Tennis Court, etc.)
Amenity and Landscape (Water Front Open Space, Gardening, etc.)

**h) Major Work Quantities**

| | | |
|---|---|---|
| Surface Excavation | | |
| Foundation Excavation | : | 140,000 $m^3$ |
| Reservoir Excavation | : | 2,000,000 $m^3$ |
| Dam Embankment | : | 160,000 $m^3$ |
| Backfilling (as Spoil Area) | : | 700,000 $m^3$ |
| Common Embankment (as Spoil Area) | : | 300,000 $m^3$ |
| Dam Concrete | : | 27,000 $m^3$ |
| Reinforced Concrete | : | 4,000 $m^3$ |
| Bridge | : | 2 bridges |
| Main Road in Park | : | L = 4,700 m, W = 20 m |

### 2.1.2 Flood Diversion Channel

**1) Structures for Short-term Project (25-year Return Period)**

The flood diversion channel for the design scale of 25-year return period is designed as the provisional flood mitigation measures for the under-mentioned flood diversion channel with the design scale of 100-year return period. The diversion channel will divert the flood runoff discharge from Tenawali Kas, Kanitawali Kas and Saidpur Kas into Kurang River (refer to Fig. C.20). In addition the flood runoff discharge from the tributary of Bedarawali Kas would be indirectly diverted through the Community Pond. The design discharges of the diversion channel and required cross section are calculated as shown in the following table (refer to Fig. C.21):

Table R C.12 Design Discharge and Required Cross Section of Flood Diversion Channel for Short-term Project

| Items | Bedarawali Kas - Tenawali Kas | Tenawali Kas - Kanitawali Kas | Kanitawali Kas - Saidpur Kas | Saidpur Kas - Ojhri Kas | Ojhri Kas - Kurang River |
|---|---|---|---|---|---|
| Design Discharge ($m^3/s$) | - | 70 | 140 | 320 | 470 |
| Channel Bed Slope | - | 1/700 | 1/700 | 1/700 | 1/700 |
| Coefficient of Roughness | - | 0.030 | 0.030 | 0.030 | 0.030 |
| Water Depth (m) | - | 4.0 | 4.5 | 6.0 | 6.0 |
| Free Board (m) | - | 1.0 | 1.0 | 1.0 | 1.0 |
| Width of Channel Bed (m) | - | 3.5 | 5.0 | 9.0 | 14.0 |
| Side Slope | - | 1 : 2 | 1 : 2 | 1 : 2 | 1 : 2 |
| Width of Channel (m) | - | 23.5 | 27.0 | 37.0 | 42.0 |

The basic concept of design is explained hereunder.

(a) The required section is estimated by uniform flow calculation using Manning's equation.

(b) The diversion channel is composed of water channel forming a trapezoidal cross section.

(c) The channel side slope is 1 : 2.0 (vertical to horizontal) without any slope protection work is adopted so as to be easily expanded to the section required for the Long-term Project.

**2) Structures for Long-term Project (100-year Return Period)**

The Long-term Project with its target completion year of 2012 would include the flood diversion channel for the design scale of 100-year return period as the structural measure continued from the above short-term Project. Through the Long-term Project, a new diversion channel will connect a channel with a flow capacity of 600 $m^3$/s from Bedarawali Kas to Tenawali Kas. (refer to Fig. C.22). Moreover, the diversion channel from Tenawali Kas to Kurang River completed in the Short-term Project will be also expanded as listed below (refer to Fig. C.21):

Table R C.13 Design Discharge and Required Cross Section of Flood Diversion Channel for Long-term Project

| Items | Bedarawali Kas - Tenawali Kas | Tenawali Kas - Kanitawali Kas | Kanitawali Kas - Saidpur Kas | Saidpur Kas - Ojhri Kas | Ojhri Kas - Kurang River |
|---|---|---|---|---|---|
| Design Discharge ($m^3$/s) | 600 | 980 | 1,120 | 1,480 | 1,790 |
| Channel Bed Slope | 1/700 | 1/700 | 1/700 | 1/700 | 1/700 |
| Coefficient of Roughness | 0.014 | 0.014 | 0.014 | 0.014 | 0.025 |
| Water Depth (m) | 4.5 | 5.0 | 6.5 | 7.0 | 7.0 |
| Free Board (m) | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Width of Channel Bed (m) | 20.0 | 22.5 | 22.5 | 22.5 | 36.0 |
| Berm (m) | 0.0 | 5.0 | 5.0 | 5.0 | 5.0 |
| Side Slope | 1 : 0.5 | 1 : 0.5 | 1 : 0.5 | 1 : 0.5 | 1 : 2 |
| Width of Channel (m) | 25.5 | 38.5 | 40.0 | 40.5 | 78.0 |

The basic concept of design is explained hereunder.

(a) The required section is estimated by uniform flow calculation using Manning's equation.

(b) The diversion channel is composed of low water channel and high water channel forming a compound trapezoidal cross section.

(c) In the city area (Bedarawali Kas - Ojhri Kas), the channel side slope is 1 : 0.5 (vertical to horizontal) is adopted so as to minimize the scale of the diversion channel. As the lining of the channel, the side slope protection such as wet stone masonry and floor concrete are designed.

(d) In the rural district (Ojhri Kas - Kurang River), the channel side slope is 1 : 2.0 (vertical to horizontal) is adopted.

**3) Cross Sectional Layout of Channel**

Khayaban - E - Johar Road, where the diversion channel is planned, is classified as 600 feet road in the CDA plan (refer to Fig. C.22). Around the center of the existing greenbelt (390 feet in width), CDA has a future plan to construct a new service road. Fig. C.22 shows a cross sectional layout plan considering this new service road programmed by CDA. But this Master Plan does not include the construction cost for the new service road.

**4) Design Features of Flood Diversion Channel**

Design features of the flood diversion channel (Route-2) are summarized as follows:

**a) Short-term Project (25-year Return Period)**

| | | |
|---|---|---|
| Catchment Area | : | 67 $km^2$ |
| Channel Bed Slope | : | 1/700 |
| Design Discharge | : | 70 - 470 $m^3/s$ |
| Channel Length | : | 7,276 m in total |
| | : | 2,150 m (Tenawali Kas - Saidpur Kas) |
| | : | 5,126 m (Saidpur Kas - Kurang River) |
| Channel Bed Width | | 3.5 - 14.0 m |
| Channel Width | | 23.5 - 42.0 m |
| Excavation | : | 1,700,000 $m^3$ |
| Side Slope Protection | : | 0 $m^2$ |
| Bridge | : | 16 bridges |

**b) Long-term Project (100-year Return Period)**

| | | |
|---|---|---|
| Catchment Area | : | 129 $km^2$ |
| Channel Bed Slope | : | 1/700 |
| Design Discharge | : | 600 - 1,790 $m^3/s$ |
| Channel Length | : | 9,726 m in total |
| | : | 2,450 m (Bedarawali Kas - Tenawali Kas) |
| | : | 2,150 m (Tenawali Kas - Saidpur Kas) |
| | : | 5,126 m (Saidpur Kas - Kurang River) |
| Channel Bed Width | : | 20.0 - 36.0 m |
| Channel Width | : | 25.5 - 78.0 m |
| Excavation | : | 4,000,000 $m^3$ |
| Side Slope Protection | : | 131,000 $m^2$ |
| Bridge | : | 20 bridges |

## 2.2 Maintenance Works

Maintenance works for the selected structures are indispensable as a part of the flood mitigation project and their plan should be prepared in due consideration of necessary work items and procedures.

### 2.2.1 Community Pond

In addition to the flood detention function, the community pond contains a potential to provide the public amenity space and improve the scenery in the urban area. Therefore, the conditions of the community pond are maintained so that the residents will easily, happily and safely access and use the area. The following items for maintenance should be considered:

**1) Securing of Detention Function**

Major works for maintenance is to secure a detention function of the community pond. In order to secure the storage capacity as well as the flow capacity of spillway/outlet facilities, required are the periodical removal of sediment, solid waste and other drifting materials from the spillway/outlet, the check dam, the oxidation pond and the flood storage pond. The weeds growing in the pond, which would affect the detention function, should be cut or pulled out periodically.

Backhoe and dump truck are required for removal of deposits from a large scale of storage ponds. A periodical inspection should be made on the damage of the spillway/outlet facilities and water leakage from the pond, and repair works should be made as required through the inspection.

**2) Safety Control**

The signboard that says “RESTRICTED AREA” or “WARNING” and fence around the off-limits area should be periodically inspected and repaired to prevent personnel from falling into the pond and trespassing into the spillway/outlet facilities and other danger zones of the facilities.

During flood, personnel in the area to be inundated should be evacuated smoothly to safer places.

**3) Sanitary Control**

The periodical inspection on the water quality of the impounding water should be made. In accordance with inspection, impounding water should be drained and pollution control measures should be taken as required. Increased waste disposal from visitors needs to be properly collected to maintain cleanliness of the park.

#### 4) Growth and Preservation of Trees/Flowers

Trees/flowers to be planted in the community pond has a significant benefit on improvement of the living environment by increased aesthetic and recreational value of the park. To keep growth and preservation of them, daily works for keeping and watering the garden are important, especially during dry season.

Considering the matters mentioned-above, the maintenance plan should be prepared specifying the places, the points and frequencies for every major facility. The plan should be well acknowledged to the competent agencies and personnel for the maintenance works, and in accordance with the plan, the maintenance works should be constantly made during a non-flooding time as well as during and after flooding time.

The concept of the maintenance plan are given as follows:

Table R C.14 Concept of Maintenance Plan for Community Pond

<table>
<tr><th>Places</th><th>Points</th><th>Frequencies</th></tr>
<tr><td>Detention Facilities</td><td></td><td rowspan="11">Inspection<br>- at least Once per Month<br>- during Flooding<br>- after Flooding<br>Repairing<br>- Time required<br>Removal of Sedimentation<br>- at least Once per Year<br>- Time required<br>Weeding and Clearing<br>- at least Once per Week<br>- Time required</td></tr>
<tr><td>Diversion Weir and Channel</td><td>Condition of Structure and Gate, Damage, Sedimentation</td></tr>
<tr><td>Dam Embankment, Crest</td><td>Slope failure, Crack, Leakage, Subsidence, Weeding, Settlement, Damage</td></tr>
<tr><td>Spillway, Spillway Bridge, Orifice for Flood Mitigation, Orifice for Drawdown, Energy Dissipater, Downstream Channel</td><td>Condition of Structure and Gate, Damage, Clogging, Sedimentation</td></tr>
<tr><td>Pond, Excavated Slope, Check Dam for Sedimentation, Oxidation Pond</td><td>Sedimentation, Slope failure, Subsidence, Weeding, Water Quality, Aquatic Plant</td></tr>
<tr><td>Facilities for Multipurpose Use</td><td></td></tr>
<tr><td>Garden, Wood Land, Open Space, Play Ground, Pond</td><td>Sedimentation, Damage, Water Quality,</td></tr>
<tr><td>Signboard, Guard Fence</td><td>Damage, Collapse</td></tr>
<tr><td>Maintenance Equipment<br>(Back hoe, Truck, etc.)</td><td>Quantity, Quality, Condition</td></tr>
</table>

### 2.2.2 Flood Diversion Channel

The flood diversion channel is designed to divert only the flood discharge over the channel flow capacity of the downstream channel but to remain the non-flood discharge within the Lai Nullah basin. Therefore, Activities of the maintenance works are concentrated in the rainy season.

The most critical issue on the maintenance works is to secure the prescribed channel flow capacity. In order to cope with the issue, required is periodical and emergency removal of sediment, solid waste and other drifting materials accumulated in the channel. Weeding of the channel is also required. A lot of drifting debris tends to accumulate and clog at the hydraulic critical points such as inlets of diversion point, piers of bridge and hydraulic drops. A special attention should be paid to those points through the periodical maintenance during non-flooding time as well as the emergency maintenance during and after flooding time. Among others, the inspection of facilities immediately after flooding will facilitate to clarify the trouble points and structural weak points, and the revision of maintenance plan should be made on the basis of the inspection.

Considering the matters mentioned-above, the concept of the maintenance plan are given as follows:

Table R C.15 Concept of Maintenance Plan for Flood Diversion Channel

<table>
<tr><th>Places</th><th>Points</th><th>Frequencies</th></tr>
<tr><td>Flood Diversion Facilities</td><td></td><td rowspan="7">Inspection<br>- at least Twice per Year<br>- during Flood<br>- after Flooding<br>Repairing<br>- Time required<br>Removal of Sedimentation<br>- at least Once per Year<br>- Time required<br>Weeding and Clearing<br>- at least Once per Week<br>- Time required</td></tr>
<tr><td>Diversion Weirs,<br>Hydraulic Drops, Bridge</td><td>Condition of Structure and Gate, Damage, Sedimentation, Clogging</td></tr>
<tr><td>Diversion Channel</td><td>Slope failure, Crack, Leakage, Subsidence, Weeding, Settlement, Damage, Sedimentation</td></tr>
<tr><td>Excavated Slope</td><td>Slope failure, Weeding, Damage</td></tr>
<tr><td>Inlet for Maintenance Flow</td><td>Condition of Structure and Gate, Damage, Clogging, Sedimentation</td></tr>
<tr><td>Maintenance Equipment<br>(Truck, etc.)</td><td>Quantity, Quality, Condition</td></tr>
</table>

## 2.3 Technical Evaluation of Selected Flood Mitigation Structures

### 2.3.1 Community Pond

The community pond gained popularity during 1980s in Japan. Many community ponds have been constructed and they proved to be effective and valuable as a flood mitigation measure in the city area. In Pakistan, the community pond is considered to be technically feasible, workable and that its operations and maintenance can be locally sustained, because a new technology is not applied and technical design such as structure type, size, construction material and equipment is not particular.

The attentions for technical design are given as follows:

(1) About the design of spillway and orifice for flood mitigation, un-gated type is adopted, because the un-gated type has no provability of flooding caused by human error under the circumstances that the arrival time of flood run-off is quite short due to the small catchment area.

(2) The foundation of the concrete gravity type spillway and embankment should be investigated by core drilling in the next stage.

(3) As the potential adverse impact, the water filled in the community pond might emit offensive odor to the neighborhoods, especially in dry-weather. However, the impact would be minimized by the proposed facilities of: (1) the bypass pipe not to allow the heavily polluted flow into the pond, (2) the intake gate to bring the less contaminated water from the adjacent river, (3) the oxidation pond to decompose the organic matter, (4) the orifice to dry up the pond, and (5) the check dam to stop the inflow of sediment and garbage.

(4) The pond with three stages made through excavation is proposed to use each stage for a recreation purpose such as garden, play ground, tennis courts and so on. The area below EL. 547.0 m including the lower and middle stages is designed to be inundated by flood less than 10-year return period probability. To minimize safety risks to use the area below EL. 547.0 m, personnel visited to the park is not allowed to enter during the rainy season.

### 2.3.2 Flood Diversion Channel

The flood diversion channel is generally designed and constructed as the principal structure for a flood mitigation measure. In Pakistan, the flood diversion is considered to be technically feasible, workable and that its operations and maintenance can be locally sustained, because a new technology is not applied and technical design such as structure type, size, construction material and equipment is not particular.

The attentions for technical design are given as follows:

(1) The flood diversion channel should be designed to divert only the flood discharge over the channel flow capacity of the downstream channel so as to secure the present natural recharging capacity to the groundwater in the downstream reaches. However, the design of the diversion structure such as type (aqueduct, siphon, etc.), scale and location are depending on the topographical condition. A feasibility study with detailed topographical survey is required to clarify the further details of the diversion structures.

(2) Approx. 4,000,000 $m^3$ of soil is excavated during construction. The excavated soil should be properly disposed of at the designated disposal area.

## 3. ON-SITE FLOOD DETENTION FACILITIES

The on-site structures will involve the various structural types, which are individually installed at each new land development sites i.e., the new residential area, commercial area, or government office quarter (refer to Fig. C.24 and Table C.1). Four (4) structural types are explained as follows:

### 3.1 Rainfall Storage Tank Installed at Individual House Lot

The rainfall storage tank is installed on the ground or in the building to collect rainwater from rooftop through roof gutters/pipes and store it so as to delay and reduce the peak runoff discharge. The standard type of the storage tank has a storage capacity of 2,000 liters, which could collect the rainfall from rooftop of 50 $m^2$ in average. Accordingly, the maximum rainfall depth to be stored in the rainfall tank is limited to only 40 mm (=2,000 liters ÷ 50 $m^2$), which is fulfilled by even 5-year return flood before its peak rainfall intensity occurs as shown in Fig R C.10.

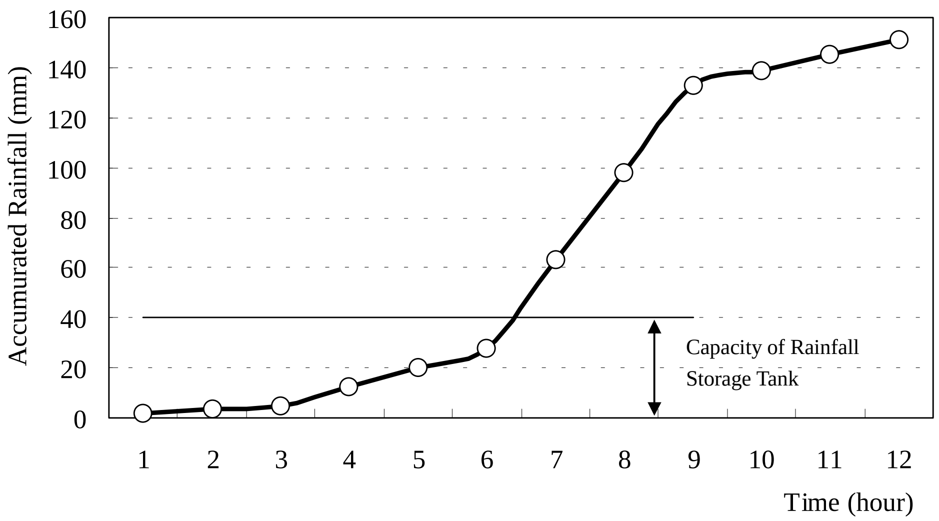


Fig. R C.10 Design Storm Rainfall of 5-year Return Period

Thus, the rainfall storage tank could hardly effect to reduce the peak runoff discharge, unless it is adopted in combination with the under-mentioned on-site flood detention pond and/or the infiltration facility. Moreover, the substantial flood mitigation effect could be achieved only when the rainfall storage tanks are installed at considerable parts of individual house lots in the basin.

Nevertheless, the rainfall storage tank has a potential function to be subsidiary water resources (the rainfall harvesting) in addition to the function of flood mitigation. In order to prevail the rainfall storage tanks, the following various expedients would be required:

(1) Dissemination of the effect of rainfall storage tank on water use among the residents;

(2) Preparation/revision of the byelaw and the Building Code to accommodate the rainfall storage tank at the individual house lot;

(3) Establishment of subsidiary system for installation of the rainwater tank; and

(4) Concession of property tax to the residents who install the rainwater tank.

### 3.2 On-site Flood Detention Pond

The on-site flood detention pond is usually placed at the downstream end of the new land development area in order to offset the increment of the peak runoff discharges inflicted by the land development. The flood regulation effect by the on-site flood detention pond could extend to both of the following middle and large-scale floods (refer to Fig. C.25):

(1) The small-scale floods (say in a rage of 5 to 10-year return period) to offset the excessive flood runoff over the flow capacity of the drainage channels immediately downstream from the land development area; and

(2) The large-scale floods (say in a rage of 25 to 100-year return period) to offset the excessive flood runoff over the flow capacity of the river channel, which is situated as the final outlet of the basin flood runoff discharges.

In order to perform the above regulation effect, the on-site flood detention ponds may have two (2) outlet holes as illustrated in Fig. C.26. The small-scale floods are discharged through only the lower outlet, while the large-scale floods are discharged through both of the lower and upper holes.

When the vacant grass land and/or the natural forest of 1 $km^2$ in extent is developed to the residential area (the moderately populated area like those in Islamabad), the probable peak discharge of 10-year return period is estimated to increase from 4 $m^3/s/km^2$ to 10 $m^3/s/km^2$ and that of 100-year return period from 16 $m^3/s/km^2$ to 23 $m^3/s/km^2$ as shown in Fig. C.25.

The necessary storage capacity of the on-site flood detention pond to offset the above increments of peak flood runoff discharge is estimated at about 150,000 $m^3$ per 1 $km^2$ of land development area assuming 4 to 7m as the average depth of pond. This on-site flood pond would have an extent of 30,000 to 50,000 $m^2$, which takes about only 3 to 5% of the entire land development area. The storage capacity as well as the extent of pond would increase in proportion to extent of the land development.

When the land development in the river basin is judged to cause the excessive peak flood runoff discharge over the flow capacity of the downstream drainage channel and/or river channel, the river administrator (or the land administrator) may be required to enforce the land developer, through bylaw, to provide the above on-site flood detention pond. Through construction of the on-site flood detention pond, the flood safety level of the river basin could be maintained irrespectively of land development in the basin.

### 3.3 Infiltration Facilities

Infiltration facilities are used to collect the rainfall and/or the flood runoff discharge and make them infiltrate into the ground so as to mitigate the flood runoff discharge. There are various types of the infiltration facilities as shown in Table C.1. The facilities are, however, applicable only to the subsurface of gravel deposits and other permeable soil. Moreover, the infiltration capacity of the facilities easily drops due to clogging by sediments, and therefore, the facility could be installed only at paved areas and green belt, where little suspended solids is yielded.

### 3.4 Flood Detention Wall at Public Open Space

The storage measure of this type is such that a public open space (such as a sport ground and a car parking area) is enclosed by a low wall with a surrounding side drain and an outlet to collect the rainfall from an entire public compound (refer to Fig. C.24). The maximum storage depth and storage time length should be limited in due consideration of the original purpose of the storage space as public utility.

Most of the facilities of this type are designed to have the maximum storage depth of 30 cm and the maximum storage time of 2 to 12 hours due to the original purpose of the storage space as the public utility. The size of the outlet should be determined on the premises that the storage will meet to the requirement of the maximum storage depth and storage time against the design hydrograph of the target design scale for urban drainage not allowing any overflow.