CHAPTER

4

HOW SHOULD IRRIGATION FACILITIES BE PLANNED AND DESIGNED?

How should irrigation facilities be planned and designed?

The irrigation facilities and the flood control facilities are the two main components of the facilities that will be implemented in the irrigation project. Irrigation facilities include intake weir, sand flushing ditch, intake gate, steep gradient main irrigation canal, sand basin, main irrigation canal, siphon or flood crossing bridge, reservoir and main drainage canal. By combining these facilities, the PMS method irrigation facilities are planned and designed to meet the following conditions:

- To secure a sufficient amount of irrigation water stably, even during drought season.
- To have all facilities function stably when anticipated flooding occurs, and to minimize facility damage even when flooding exceeds the envisaged scale.
- To avoid failure of water intake and conveyance to the intake gate and main irrigation canal due to sedimentation/blockage.
- To give full consideration to the social environment in the process of acquiring land for the main irrigation canal, planning for reservoir and sand basin, and in distributing water.
- To keep economic construction with good workability in mind when planning the facility location avoiding rock excavation and large-scale excavation/land reclamation work as much as possible.
- To ensure that local residents can operate, maintain and manage the facilities sustainably.

Each intake facility is designed as follows, with a structure to facilitate maintenance and management by local residents using locally procurable boulders, bricks, timber, iron plates/ annealing wires, cement and soil.

- For the intake weir, the size of stones comprising the weir body is properly dimensioned, wings are fully protected and the weir height is minimized to prevent being swept away. Moreover, to maintain the intake water level and the water depth of the main irrigation canal even during drought season, the weir height is secured to ensure sufficient amount of irrigation water reaches the irrigation beneficiary area.
- For the sand flushing ditch, a cross-sectional area and flow velocity allowing sediments to be discharged are secured to avoid the area upstream of the intake weir from being buried by sediments.
- For the intake gate, its function of taking and adjusting the amount of water required for irrigation with a stable structure against water pressure from the river and sediment load is ensured.
- The cross-sectional shape and revetment structures of the steep gradient main irrigation canal and main irrigation canal shall be stable against the inner hydraulic pressure and external earth pressure by securing a certain flow velocity and ensuring no sediments are deposited in the canal.
- The sand basin structure shall ensure that deposited sediments can be easily discharged. (sand drainage canal, sand drain gate)
- Since constructing the reservoir involves large-scale embankment works, the stability of reservoir embankment is secured by taking seepage water countermeasures including the application of silty cray on the internal slope surface of the reservoir as fully treating the foundation.
- A certain flow velocity is secured in the siphon to prevent sands from accumulating in the canal, and a safe underground culvert structure against the vertical load and earth pressure shall be provided. The flood crossing bridge shall be secured the width for flowing down the flood water.
- The main drainage canal is capable of promptly discharging excessive water from the irrigation beneficiary areas to the river. Such like the main irrigation canal, the cross-sectional shape and revetment structures shall secure the stable condition against the internal water pressure and external earth pressure.

The following pages give commentary on the above contents:

4.1 Layout planning and Design Process of Irrigation Facilities

4.1.1 | Layout Planning of Irrigation Facilities

Based on the basic concept jointly established with local residents and other concerning parties and the river survey result, a layout planning for irrigation facilities of the PMS method irrigation facilities, that are, intake weir, intake gate, steep gradient main irrigation canal, main irrigation canal, sand basin, reservoir, siphon and main drainage canal. These irrigation facilities shall, in principle, ensure the sufficient water head that allows to flow irrigation water by gravity from candidate intake site to irrigation beneficiary area. Accordingly, the layout planning is established by the following process :

Start

[Checking the existing status]
Existing cultivated lands, intake point and irrigation canals are checked

[Selecting irrigation beneficiary areas]
• Discussions with farmers are organized to decide on the target irrigation beneficiary areas, including new irrigation areas.
• The point conveying water from the new intake site (near existing intake site) is selected.

[Layout planning of the steep gradient main irrigation canal and the intake site]
• The intake site shall be set at a point where the required hydraulic gradient can be secured. The steep gradient main irrigation canal shall be arranged by the shortest distance to the target water conveyance areas.
• Suitable land for intake site is determined by gradient, river channel curve, bedrock, sand bar and other elements.

See Chapters 2 and 3

[Locating the sand basin and reservoir]
• The sand basin is provided in a location around the nearest existing intake site from the new intake site and near the river. In the case multiple intake sites are integrated into one line with one intake location, a regulating pond will be installed near the original existing intake sites in the downstream section of the sand basin, and it will be a diversion point and a junction point with the new irrigation canal.

[Layout planning of the main irrigation canal (including reservoir, siphon, etc.)]
• The main irrigation canal is provided in the downstream section of sand basin.
• A siphon or flood crossing bridge shall be provided in case a canal is likely to cross other waterways, rivers, streams and wadi. And the reservoir shall be provided in case a canal is likely be damaged due to flash floods and debris flows from slopes and tributaries.

[Layout planning of main drainage canal]
• The main drainage canal is provided on a low elevation ground where drainage is required due to tendency to be damped.

See Chapter 4

End

Figure 4.1 Workflow of the Layout Planning for Irrigation Facilities[2)]

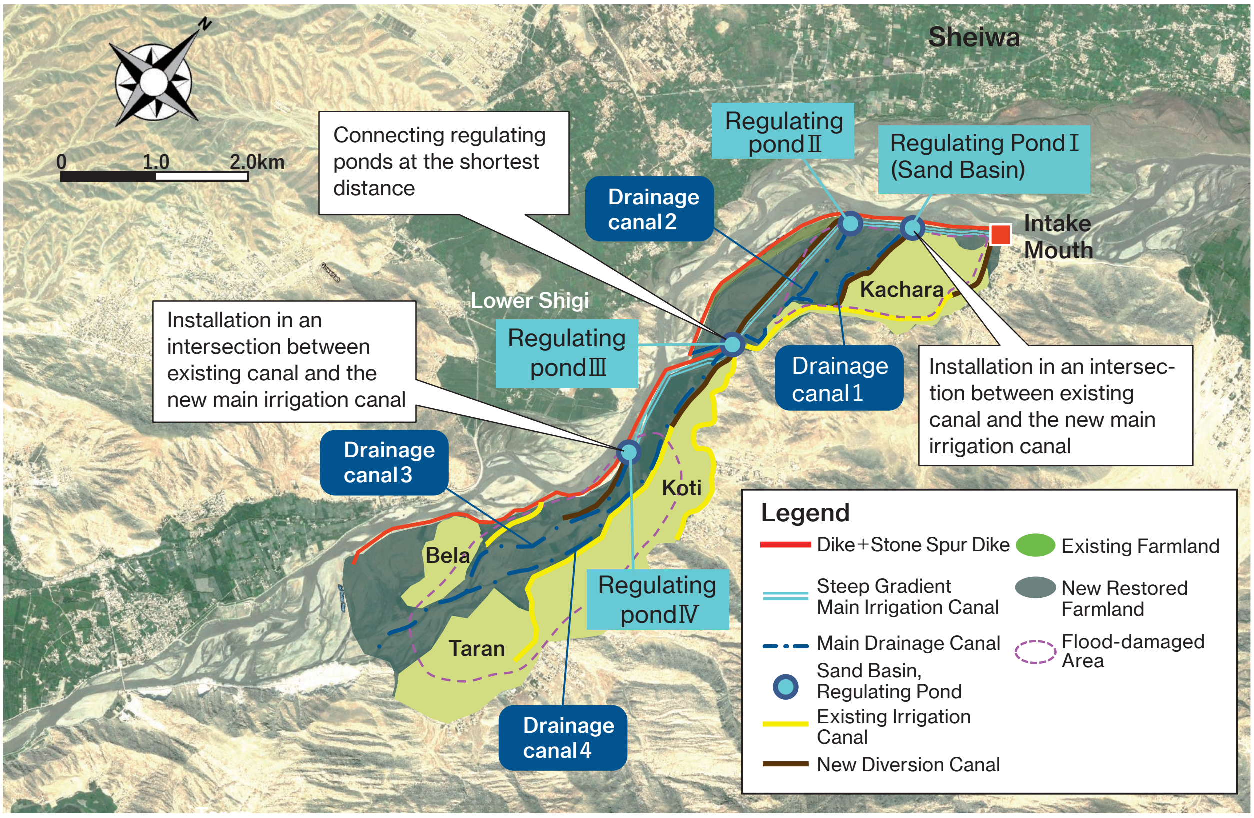


Figure 4.2 Examples of Layout Planning for the Intake Sites, Sand Basin and Main Irrigation/Drainage Canal[2)]

(1) Layout Planning for Intake Weir and Intake Gate

The intake weir/gate location is considered based on the status of the river channel alignment, sand bar, riverbed, riverbank and other river conditions around the candidate intake sites. Based on whether the stability of the weir body has been secured, water utilization is prevented due to sediment deposition and any problems with workability and other perspectives occur, the optimum location of the intake weir and gate is decided. The locations where rock distribution may hinder construction work or where sand bars have formed near the riverbank and may well block the intake shall be avoided. In addition, in large-scale rivers, it shall be confirmed that the possibility of securing a flow path for flood divergency by dividing the river channel by sandbars.

(2) Configuration for the Steep Gradient Main Irrigation Canal and Main Irrigation Canal

The steep gradient main irrigation canal conveys suspended and wash loads contained in the irrigation water from the intake gate to the sand basin without depositing them in the canal while the main irrigation canal carries irrigation water from the sand basin to the beneficiary irrigation area, which shall be routed to go from the sand basin to the starting point of the existing canal as directly as possible (preferably linear). Both canals shall be routed considering the status of land use (private land, farmland, etc.) and obstacles (other canals, buildings, roads, rock distribution, uneven geographical features, etc.) and avoiding such conditions.

(3) Location of the Sand Basin

The sand basin is a facility which deposits and drains sediment having flown in through the steep gradient main irrigation canal. It shall be located at the end of the steep gradient main irrigation canal and within 1 km or so from the intake gate, considering the topographic gradient, canal gradient, sand basin depth and other aspects. Since sands and water in the sand basin are removed to the river via a drain gate, the sand basin is located as close to the river as possible and a (sand) drainage canal shall be provided between the sand basin and the river. In many cases, the sand basin is also utilized as a regulating pond as described below.

(4) Location of the Reservoir

In cases where a small slope with relatively steep gradient is located right next to the steep gradient main irrigation canal and the main irrigation canal routes and the canals are prone to damage from flash floods and debris flows, a reservoir shall be provided. However, no reservoirs should be installed in any large-scale valleys.

(5) Location of the Siphon

In cases where the main irrigation canal traversing other waterways, rivers, stream, wadi and other elements is inevitable, an (inverted) siphon or flood crossing bridge shall be provided to protect the canal from flash floods and debris flows.

(6) Location of the Regulating Pond

A regulating pond is provided at the intersection between the proposed main irrigation canal and existing canal in order to remove suspended and wash loads. At the same time, transmission gate and diversion gate are provided to distribute the water properly. If the main irrigation canal is not so long, sand basin is utilized as a regulating pond.

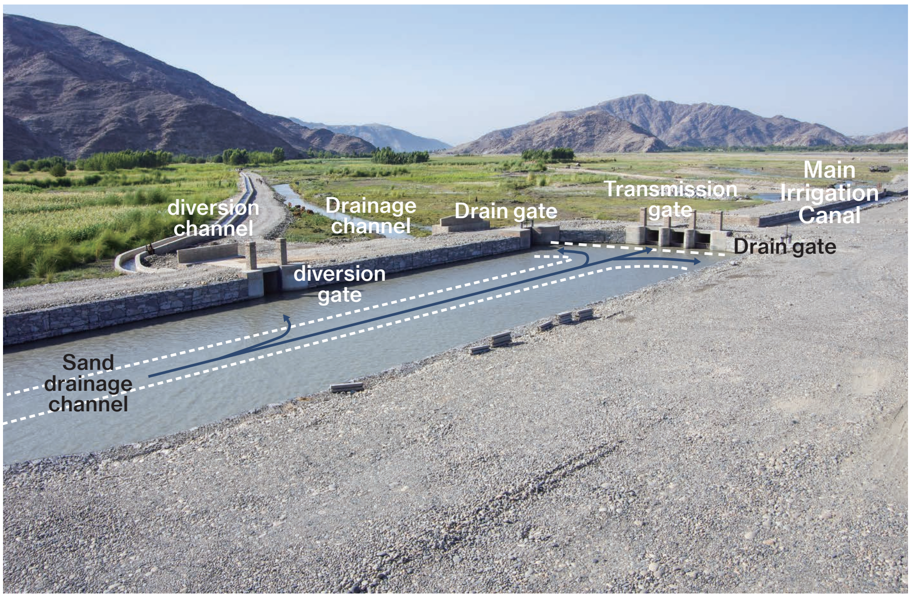


Photo 4.1 Regulating Pond from the Upstream Side (1)

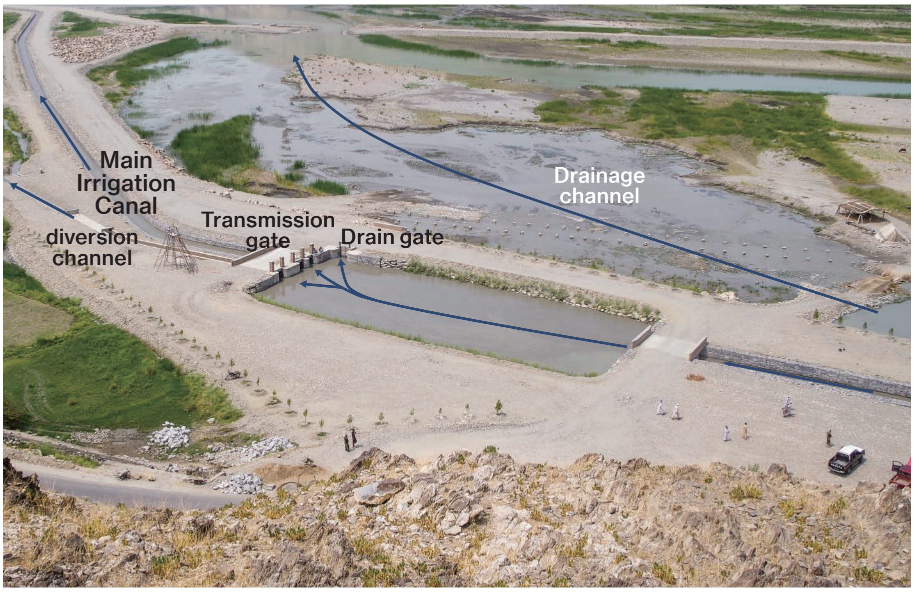


Photo 4.2 Regulating Pond from the Upstream Side (2)

(7) Layout Planning for the Main Drainage Canal

The main drainage canal is a facility which drains off excess water and rainwater from irrigated farmland to the river promptly without retaining them to prevent water damage and significantly extend the cultivation area. Centered on the main drainage canal, it shall be planned as a drainage network covering the whole area and ensuring the drain water flows from the existing/new drainage canal networks to the main drainage canal. The main drainage canal route shall be arranged at a lower ground level where the topography is depressed compared to the surrounding area.

Based on the arrangement shown above, a project briefing is organized for residents residing near the steep gradient main irrigation canal/main irrigation canal, sand basin and other facilities to be constructed to discuss the agreement on land acquisition with them, the lease for constructing the aforementioned structures and the layout planning. The discussion with residents will address the specific scope area and period of the land acquisition and lease during construction separately once the facility design and implementation schedule are clarified.

4.1.2 | Design Process of Irrigation Facilities

The PMS method irrigation facilities, namely the intake weir, intake gate, steep gradient main irrigation canal, main irrigation canal, sand basin, reservoir, siphon and main drainage canal, shall be designed following the design process as shown in Figure 4.3. The approach to design each facility is detailed from Sections 4.2 to 4.6. Canals like the steep gradient main irrigation canal, main irrigation canal and main drainage canal shall ensure a certain flow velocity which is exceeded to prevent suspended load sedimentation but also limit the flow velocity to prevent canal section erosion. Although a sufficient cross-sectional area shall be secured to ensure the required amount of irrigation water can be discharged, the flow rate varies widely according to the canal gradient which is generally related to a factor of the ground elevation where the canal is located. Moreover, the feasibility of acquiring land, economic efficiency and workability of the canal route shall also be considered. Referring to the Manning equation, the various trade-offs for such canal design can be summarized as follows:

$$V = \frac{1}{n} \times R^{\frac{2}{3}} \times I^{\frac{1}{2}}, \quad Q = A \times V \qquad (4.1)$$ [2), see 11)]

Here, Q: flow rate (m^3/s), V: flow velocity (m/s), n: roughness coefficient, A: cross-section area (m^2), R: hydraulic radius (m) (R = A/S), S: wetted perimeter (m), I: riverbed gradient

- Assuming that the cross-sectional area and flow velocity maintain a certain level (flow rate), when the cross-section is designed to be vertically longer by increasing the design water depth of the canal, a steep canal gradient is required, given the extended wetted perimeter and the reduced hydraulic radius. When the cross-section is designed to be horizontally longer by decreasing the design water depth of the canal, the canal gradient can be gentle since the wetted perimeter becomes shorter and the hydraulic radius is extended.
- If the ground gradient is gentle, a horizontally-long cross-section with excavated canal shape is preferable with workability and economic efficiency in mind, since a vertically-long cross-section requires a dike. If the ground gradient is steep, however, a vertically-long cross-section is economically more efficient since the excavated soil amount will generally increase for a horizontally-long cross-section.
- The land area to be acquired increases in amount if the canal shape is widened and vice versa. To narrow the canal shape, the water level gradient must increase. In turn, the weir height must be heightened which raises the risk of flooding and pushes up the construction cost.

The irrigation water is temporary stored in a sand basin and reservoir to capture suspended loads and distribute the water over irrigated farmland. If increasing the reservoir depth to secure a certain amount of storage water, the land area to be acquired declines and is more easily managed. Conversely, reducing the depth makes the bottom of the reservoir more accessible and more easily managed during dredging while enlarging the land area to be acquired.

As above, when designing the ground layout plan of the intake weir and gate, main irrigation canal, main drainage canal, sand basin and reservoir and designing the specifications, profile and cross-section, secure the required functions by considering various trade-offs as well as constraints on land acquisition, workability, economic efficiency and manageability of the facility.

In designing a series of facilities constructed from the intake site to the irrigation beneficiary area, the difference in elevation between the intake site and irrigation beneficiary area is particularly crucial to secure the head. If the weir height is lowered to stabilize the weir body, the water level at the intake and main irrigation canal declines, meaning the cross-section has to be horizontally extended. This may increase the land area having to be acquired and the earth extraction, according to the surrounding topography, adversely affecting both workability and economic efficiency. Conversely, when introducing a vertically-long cross-section by limiting the land area and earth amount, the required head must be secured by boosting the weir height to ensure the water level remains high at the intake and main irrigation canal. In some cases, the intake site must be rearranged upstream.

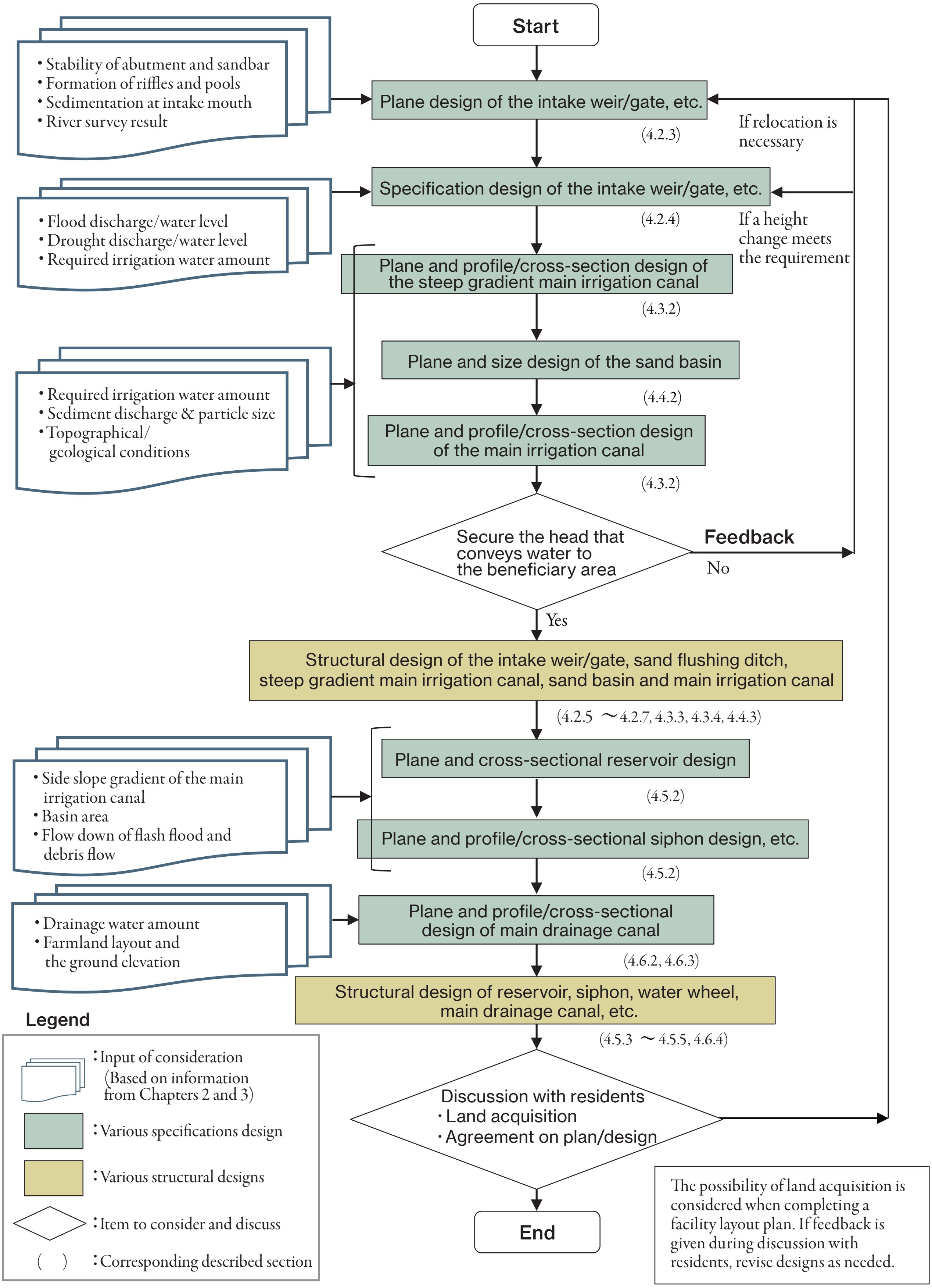


Figure 4.3 Design Process of Irrigation Facilities[2)]

4.2 Design of the Intake Weir and Intak Gate

4.2.1 | Basic Design Policy

When designing the intake weir and intake gate, the following key matters must be avoided. The design process of the intake weir and intake gate is shown in Figure 4.4

- The intake gate shall not be located in a place prone to sedimentation in river channels. This is to avoid intake failure due to sedimentation in the intake gate.
- The plain shape of oblique weir is not a straight alignment but a curved shape that is concave toward the downstream. This is because the water flowing down from multiple directions of the concave curved weir are collected in the center downstream of the weir to offset the flow energy and preventing the opposite side bank from scouring.
- The height of the intake weir shall not increase excessively. This is because the higher the weir, the greater the risk of collapse due to river flow when flooding intensifies.
- The apron immediately downstream of the intake weir shall not have a steep slope. This is to prevent erosion as much as possible, since the apron is the most prone to erosion caused by the water flow.

4.2.2 | Type of the Intake Weir: Boulder Oblique Weir

(1) Adoption of the Boulder Oblique Weir

The intake weir is a structure designed to secure the intake level by raising the level of the river. In the PMS method, a boulder curvilinear oblique weir is adopted as the intake weir; constructed as a convex shape toward upstream and diagonally toward downstream by piling up boulders to construct the weir body and reinforcing its attachment part to sand bars using boulders and cobblestones.

The boulder oblique weir is arranged as follows to secure its stability during a flood. Accordingly, its features include anti-erosion against flooding, allowing water to be taken in stably, even during drought seasons. Moreover, the construction remains affordable and it is easy for local residents to maintain and manage. Accordingly, the boulder oblique weir is one of the key highlight features in the PMS method irrigation project.

Consideration of a plane form for curvilinear oblique weir is processed under the course of types: a) simple groin, b) full width weir at right angles to the center of the stream flow, c) linear oblique weir and d) curvilinear oblique weir as shown in Figure 4.5. Each concept has the following characteristics:

a. Simple groin: a groin is built out from one bank, which may cause scouring of the portion immediately downstream of the groin edge that hinders efforts to secure the intake level.

b. Full width weir at right angles to the center of the stream flow : the weir body is constructed over the full width of the river at right angles to the center of the river. Compared to (a), the intake level is surely secured while the load to weir body is greater given the large unit width discharge.

c. Linear oblique weir: the weir is provided diagonally toward the flow direction. The weir width exceeds (b), which allows the unit width discharge to be controlled and mitigates loads on the weir body. Meanwhile, as shown in (c) of Figure 4.5, overflow from the weir may scour the river banks and sand bars eroding the weir body part attached to them.

d. Curvilinear oblique weir: as shown in (d) of Figure 4.5, this weir curves around the shape of the linear oblique weir as shown in (c) as above and is wider than the linear oblique weir. The large weir width means a longer overflow length while reducing the unit width discharge at the weir crest. This reduces the tractive force on the weir body and secures weir body stability. Moreover, a long overflow length can also effectively control any fluctuation in water level caused by changing the river flow discharge. Further, strong flows from the sand flushing ditch and spillway and overflows from top the weir gather in the center of the curved part and the flow velocity is reduced by setting off flow energy to prevent erosion of the weir body part attached to the river banks or sand bars.

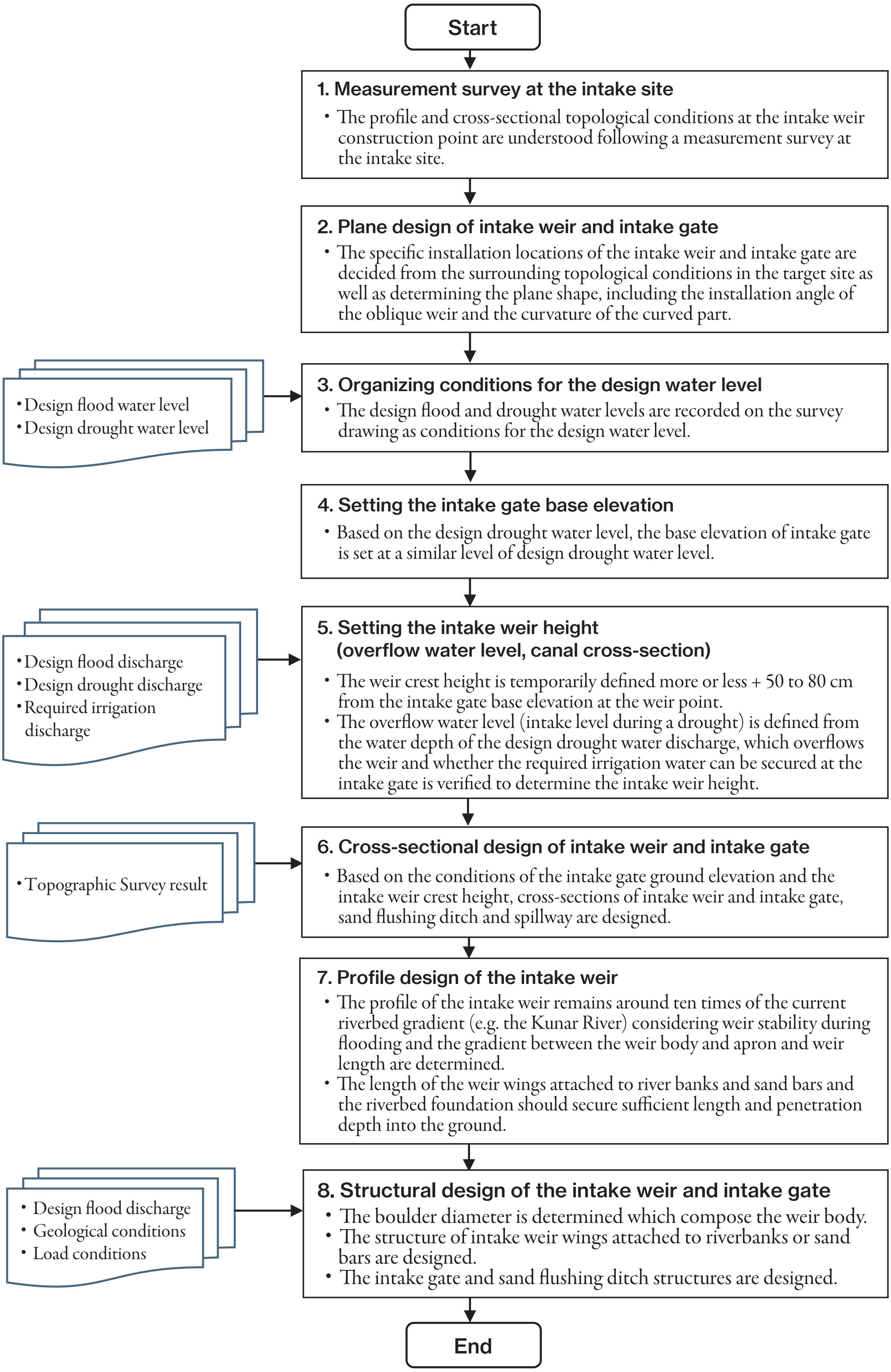


Figure 4.4 Design Process of Intake Weir and Intake Gate 2)

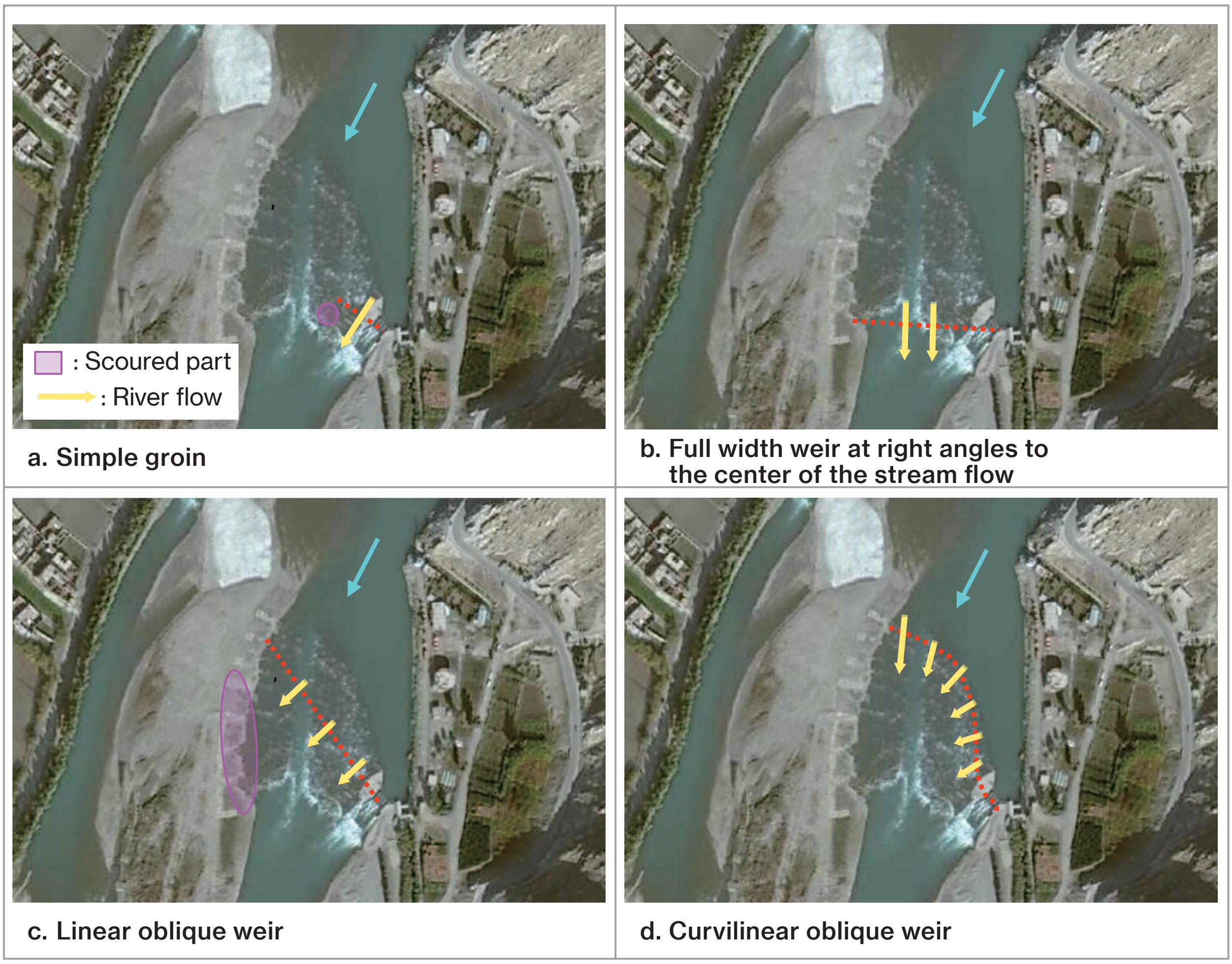
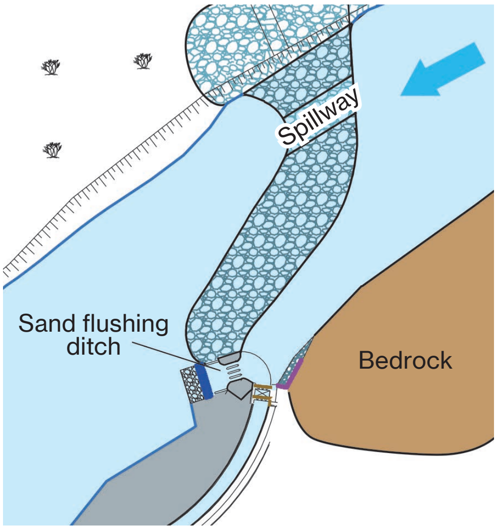
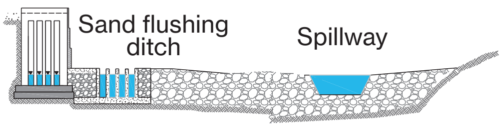
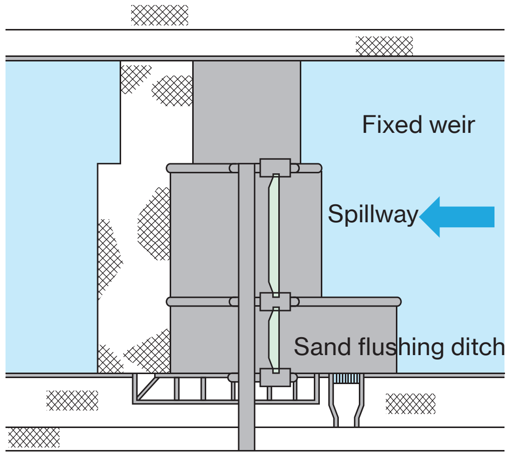
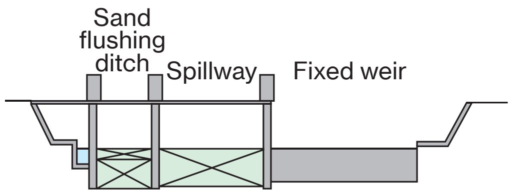


Figure 4.5 How to Consider to Select the Plane Type for the Intake Weir [2)]

(2) Comparison between the Boulder Oblique Weir and Concrete-fixed Weir/Sluicegate Weir

The type of the intake weir combining a fixed weir with a reinforced concrete and sluice gate is adopted in many countries and usually constructed at right angles to downstream direction. The common features of the intake weir and boulder oblique weir and their applicability to Afghanistan are shown in Table 4.1, suggesting that the boulder oblique weir remains suitable in Afghanistan.

Table 4.1 Comparison of the Intake Weir Types [2)]

Item	Boulder Oblique Weir (adopted in PMS method irrigation project)	Concrete-fixed weir/sluice gate (frequently adopted in irrigation projects in many countries)
Outline drawing	 	 
Features	• The weir is provided diagonally to the flow direction and will have a convex shape toward the upstream. • The weir body is constructed by piling up boulders on the riverbed and reinforcing the wings with boulders and cobblestones.	• A fixed weir and sluice gate are provided to traverse the river and raise the water level. • Since it is usually constructed at right angles to the river flow, it shall be a solid concrete structure to withstand the flow velocity.
Merits	• The weir is provided diagonally to the river flow so that flow energy can be reduced. • The weir body also functions as a training dike wall, smoothly conveying river water into the intake. • The construction is relatively affordable.	• It can withstand the flow energy thanks to the concrete structure. • The concrete structure needs comparatively little maintenance and management.
Due diligence points	• Regular maintenance and management is required, such as additional input of stone materials.	• A concrete structure is time-consuming and costly to repair once damaged. • The sluice gate and other facilities must be sufficiently maintained and managed. Once damaged, repairs are time-consuming, costly and almost impossible for residents to do so alone. • Construction is relatively costly.
Applicability to Afghanistan	• Easy for local residents to maintain and manage. • It is a structure applying stone materials which are abundant in many regions. • There are many PMS method irrigation project cases.	• Maintenance and challenging for local residents.

(3) Comparison between the Boulder Oblique Weir and Conventional Intake Facilities

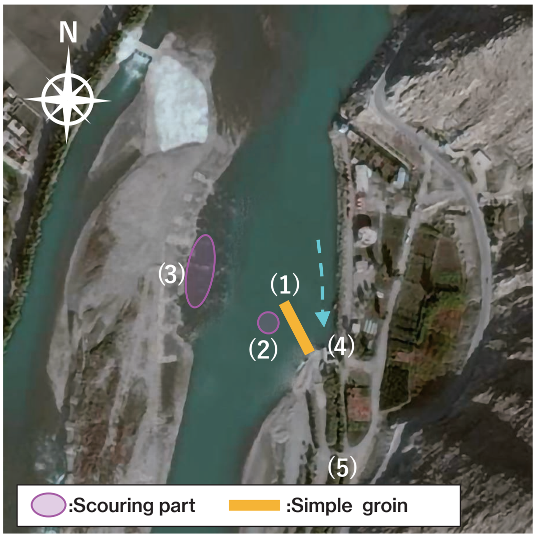
In many cases, when water is drawn from rivers in Afghanistan, the following difficulties emerge:

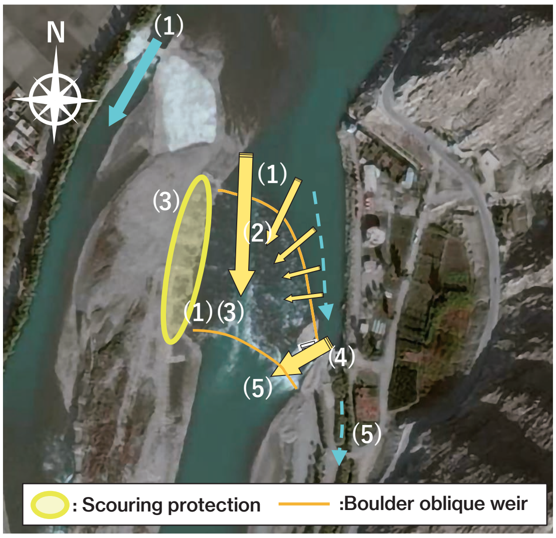
- The intake gate is vulnerable to flooding.
- There is considerable sediment inflow while flooding.
- Stable water intake is difficult to ensure during flood and drought seasons.

Focusing on these difficulties, Table 4.2 compares intake facilities by PMS centered on the boulder oblique weir and conventional intake facilities. Many of the latter have a simple groyne form, the riverbed of which often declines at the edge and seems incapable of securing the intake level. Moreover, amid unchanged intake amount, the water intake function may not be available during a flood season and it may be vulnerable to flood and sediment inflow. Meanwhile, the PMS method irrigation project overcomes such challenges by developing and adopting a unique, simple and practical intake system (boulder oblique weir, intake gate, steep gradient main irrigation canal and sand basin).

Table 4.2 Comparison between the Boulder Oblique Weir and Conventional Intake Facilities [2)]

Conventional Intake Weir (Simple groin)	Boulder Oblique Weir in PMS Method Irrigation Project





Issues

(1)	• There is concern about possibility of washout or destruction of the weir itself.
(2)	• It is difficult to secure the water intake level because the weir body is broken due to riverbed decline by scouring at the tip of the groin.
(3)	• River water route moves due to opposite side bank scouring, and the water cannot flow into the intake mouth.
(4)	• There is no function to adjust intake water amount at the intake mouth.
(5)	• Flood flows directly into the irrigation canal. • There is a lot of sediment inflow into the irrigation canal. • There is no sand settling function for the extracted water, and there is a lot of sediment accumulation in the main irrigation canal.

Countermeasure

(1)	• Flood diversion channel is secured by dividing the river channel by sand bar. • Curved full-width weir reduces unit width overflow discharge.
(2)	• The entire width and gentle gradient of weir body from the riverbank to the sandbar prevents scouring at the immediately downstream tip of the weir and secures the intake water level.
(3)	• The strong flows from sand flushing ditch, flood spillway and overflow from top of the weir are collected in the center downstream of the curved weir to offset the flow energy and prevent scouring on the opposite side bank.
(4)	• The intake water amount can be adjusted manually with the double flush board method at intake gate and sand flushing ditch.
(5)	• Flush board method prevents flood flow from flowing into the irrigation canal. • Extracting overflown water above the flush board can prevent the inflow of sediment to the main irrigation canal. • Sediment inflow and deposition in the main irrigation canal can be prevented by providing a sand flushing ditch and a sand basin.

(4) Basic Policy of the Boulder Oblique Weir Design

According to the actual construction in the existing PMS irrigation project, the specifications of the boulder oblique weir are as follows. Benchmarking these, the boulder oblique weir is designed in line with regional conditions. The following three types and specifications will be the standard designs in the PMS method irrigation project: 1) a boulder oblique weir is introduced as an intake weir securing a wider weir width, 2) the weir wings attached to the river banks and sand bars are reinforced by boulders and cobblestones and 3) the intake weir is provided diagonally; curbing against the river flow. The height, length and stone size of the intake weir are determined according to the design flood discharge, required irrigation water, locally available size and amount of boulders and cobblestones and other regional conditions. The design of the boulder oblique weir follows Figure 4.6, Figure 4.7 and the following items.

- Intake weir plane design：	The axis direction of weir body is provided at an angle of 30 to 45 degrees diagonal to the river flow. The weir width is twice to three times of river width and in convex shape to the upstream direction.
- Intake weir profile design：	The weir height is around two meters or lower. The apron gradient and weir height are set as 2.0% or lower.
- Intake weir cross-sectional design：	The intake weir crest elevation is set at around 0.5 to 0.8 m higher than the base elevation of intake gate. The base elevation of intake gate follows the design drought water level. The base elevation of sand flushing ditch is around 0.7 to 1.0 m lower than the intake gate elevation. The secured width of spillway is 10 to 20 m and depth is around 1m.
- Intake weir structural design：	The size of boulders comprising the intake weir body is 0.5 to 2.0 m.

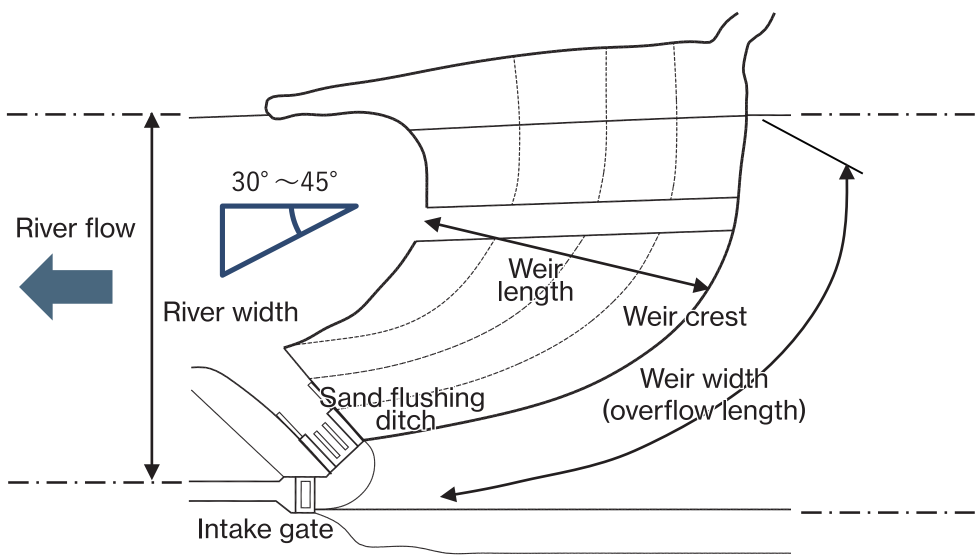


Figure 4.6 Plane Design Model of the Boulder Oblique Weir [2)]

- Protection of wings and foundations of the weir body：	Abutments (wings) are protected using boulders. The weir foundation shall be embedded below the riverbed.

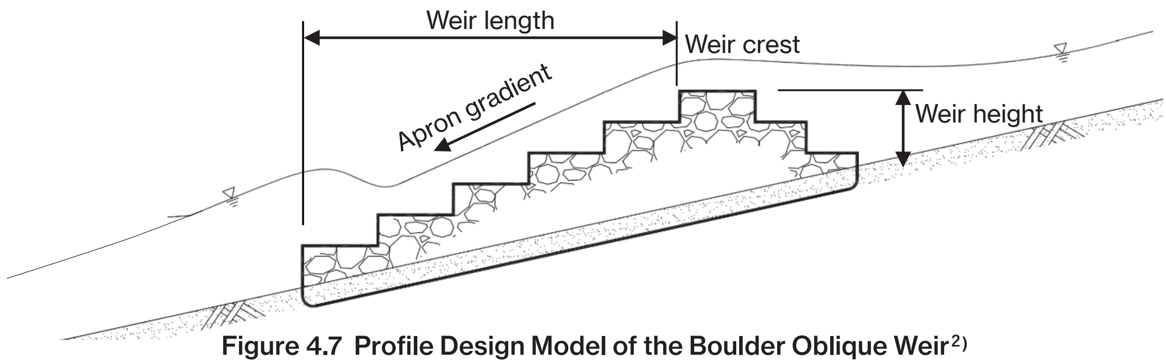


Figure 4.7 Profile Design Model of the Boulder Oblique Weir[2)]

4.2.3 | Plane Design of the Intake Weir and Intake Gate

(1) Location of the Intake Weir

The intake weir location is decided from among several candidate points considering the following:

- Whether sand bars and riverbanks attached to the wings of the intake weir body are stable and whether the upstream and downstream lengths of the attaching parts to be reinforced are sufficiently secured.
- If it is located immediately downstream of the rock and is suitable in terms of both stability and workability against flooding.
- Whether there are such concerns that the intake is blocked, including the distribution of sand bars near the planned construction site of the intake gate.

The above points are explained in Figure 4.8 as a pattern diagram. Since the weir part at opposite bank is attached near the edge of the sand bars in Location 1, the sand bar is likely be eroded by flooding, damaging the stability of the weir body from its attaching part. Moreover, its workability is low since constructing the intake gate and sand flushing ditch involves rock excavation. Sand bars are distributed near the planned intake gate site in Location 3 and sedimentation raises concerns. Accordingly, Location 2 is the most suitable for installing the intake weir.

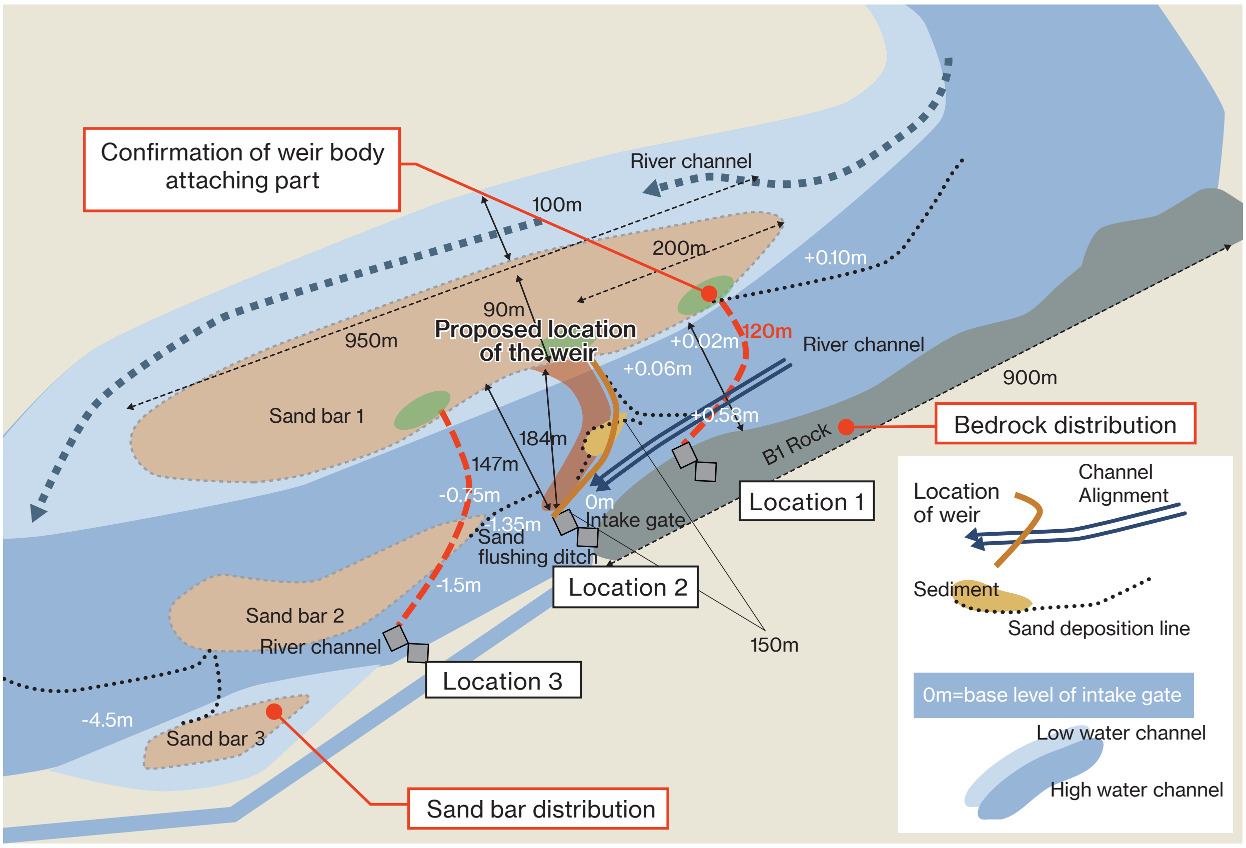


Figure 4.8 Example for Setting the Location of the Boulder Oblique Weir [4)]

Those matters to be noted when constructing an intake weir in a river channel with unstable flow reference the Miran intake weir as shown in Column 4-1. In the Miran intake weir location, the original main channel was blocked by sedimentation and the riverbank eroded by the other newly formed main channel. The intake had to be relocated several times due to changing river channels, which made it difficult to define a proper location. In such cases, the following measures allow the intake weir to be provided where the main channel is fixed to some extent:

- A stone spur dike is constructed at the river curved part along the newly formed main channel.
- Former main channel blocked by sedimentation is dredged and recovered to control the water flow into the current main channel.
- A boulder oblique weir with sand bars between the former and current main channels as its wings is constructed by reinforcing the wings and integrating sand bars and weirs.

As above, the intake weir can be provided in those instable river channels by preventing riverbank erosion, fixing the current main channel, stabilizing sand bars and fixing the intake.

The whole plane diagram of the Miran Weir and surrounding protection as planned under the above policy are shown in Figure 4.9.

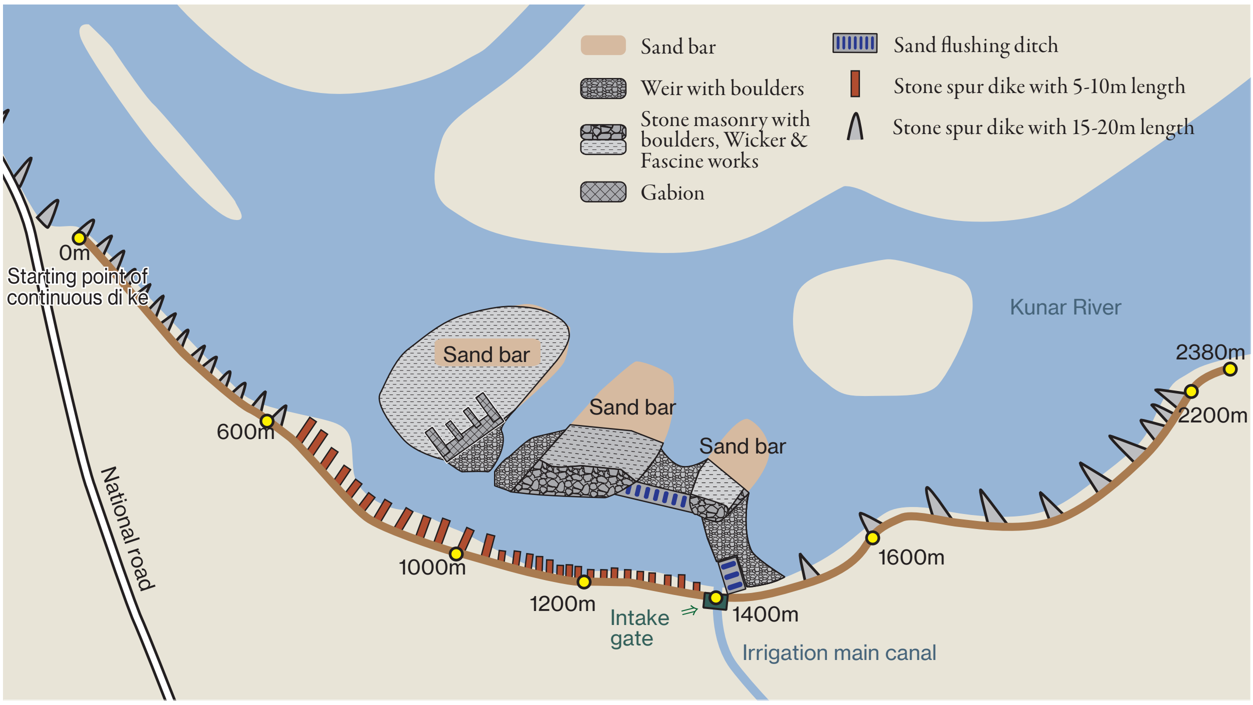


Figure 4.9 The Whole Plane Diagram of the Miran Weir[2)]

Text Block 4-1: Background to the Plane Design of the Miran Intake Weir

■Changing intake location

The riverbank of the Kunar River in Behsud Province is prone to severe flooding. In the decade since 2003, the intake location was changed three times: Intake A (2004) ⇨Intake B (2005) ⇨ Intake C (2010) ⇨ Intake D (2013). This was due to large floods destroying the intake and causing the main channel to be relocated.

■Background to the changing flow directions

- Consecutive dikes were deeply eroded, causing the emergence of new river channels (i) along the dikes.
- Sedimentation occurs in river channels (iii) to (v) due to significant flooding in 2010 and 2013, increasing the flow rate in the channel (ii).
- Straight channels (i) and (ii) are merged with the increased flow rate, forming a new main channel.

■Countermeasures

The natural flow before engineering intervention shall be recovered as much as possible.

1. After recovering the flow rate in the former natural channels (iv) and (v) by river channel excavation and returning the main channel to its original natural channel, new channels (i) and (ii), whose flow rate was reduced, were blocked. By taking the revetment line in 2013 as the design dike line, erosion of farmlands by changing main channels was prevented.
2. To prevent the expansion of new main channels (i) and (ii), the intake weir was constructed as a disaster prevention facility.

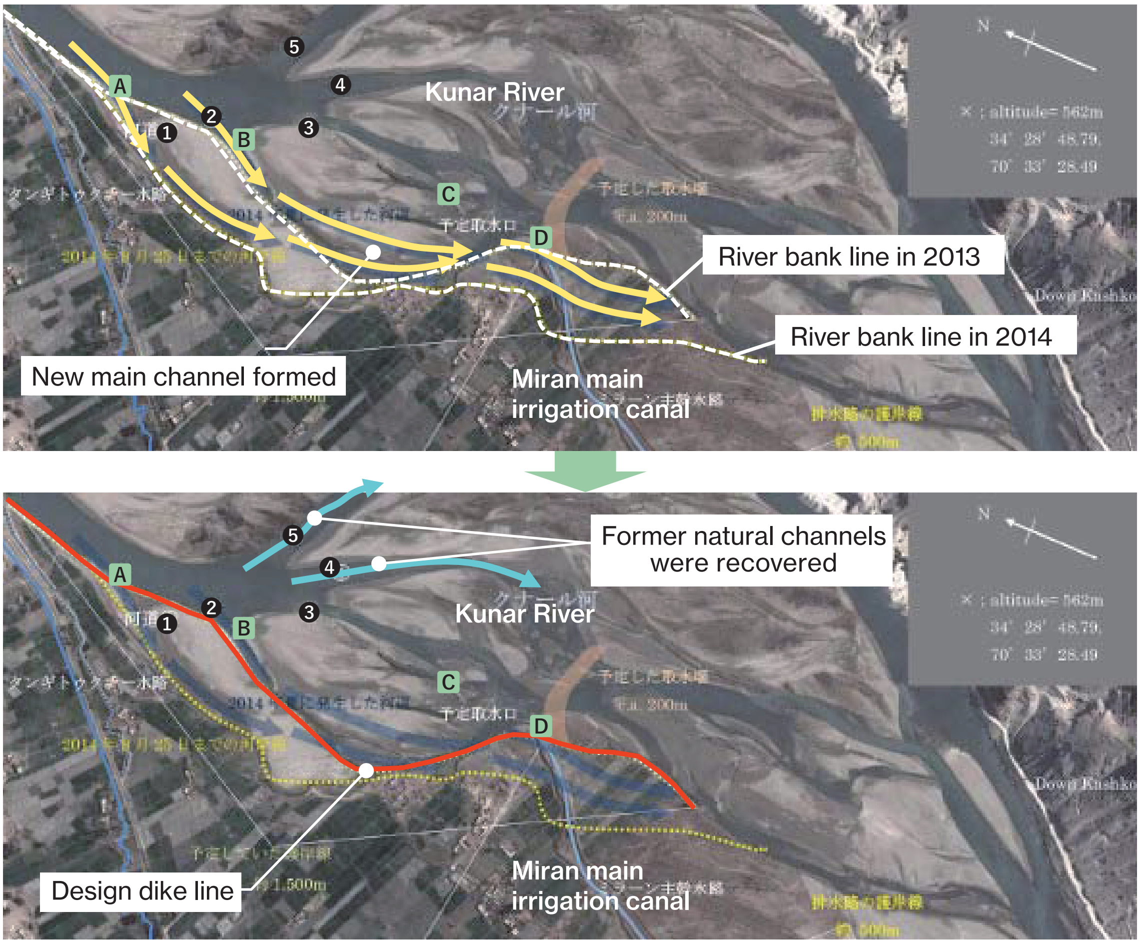


Figure Riverbank Erosion and Countermeasures to the Intake Gate in Miran [4)]

(2) Plane Design of the Boulder Oblique Weir

The plane design of the intake weir in PMS method irrigation facilities are as follows:

- The intake weir type shall be a boulder oblique weir, the body of which is constructed by stone masonry of boulders.
- To stabilize the weir body by controlling the river flow overflowing the weir for each unit width discharge, the weir is provided diagonally to the flow and a weir width twice to three times as long as the river width is secured.
- The plane alignment of the overflowing part is shaped convex relative to upstream and its curvature radius shall be around 70 to 150 m. The larger the curvature radius, the shorter the weir width, or vice versa.
- The average weir installation angle is 30 to 45 degrees to the flow. The larger the angle, the shorter the weir width, or vice versa.
- While the weir width is determined by several elements, including the curvature radius and flow angle as above, a weir width capable of securing weir body stability shall be set when applying unit width discharge during the overflow as defined by the weir width to "4.2.5 (4) Confirmation of the Stability of Stone Materials composing the Boulder Oblique Weir".
- The size of boulders comprising the weir body is based on particles of 0.5 to 2.0 m which can remain in place, despite the tractive force of the design flood water discharge. The validity of the boulder size is evaluated by "4.2.5 (4) Confirmation of the Stability of Stone Materials composing Boulder Oblique Weir" as shown below.

If boulders of the required size are unavailable around the site, the required boulder sizes can be reduced to a certain extent by reducing the overflow amount per unit width, lengthening the weir width, reducing the weir installation angle, changing the curvature radius of the curved part or other measures. Moreover, the oblique weir, due to its form features, has a weir length around the sand flushing ditch near the intake gate shorter than the length of the opposite bank and the gradients of the weir body and apron steepen. Accordingly, the length of the weir parts attached to the opposite riverbank and sandbar shall be as long as possible to stabilize the weir body, and spillway shall be provided to prevent excessive concentration of running water on the intake gate side. Figure 4.10 shows a plane drawing of the boulder oblique weir.

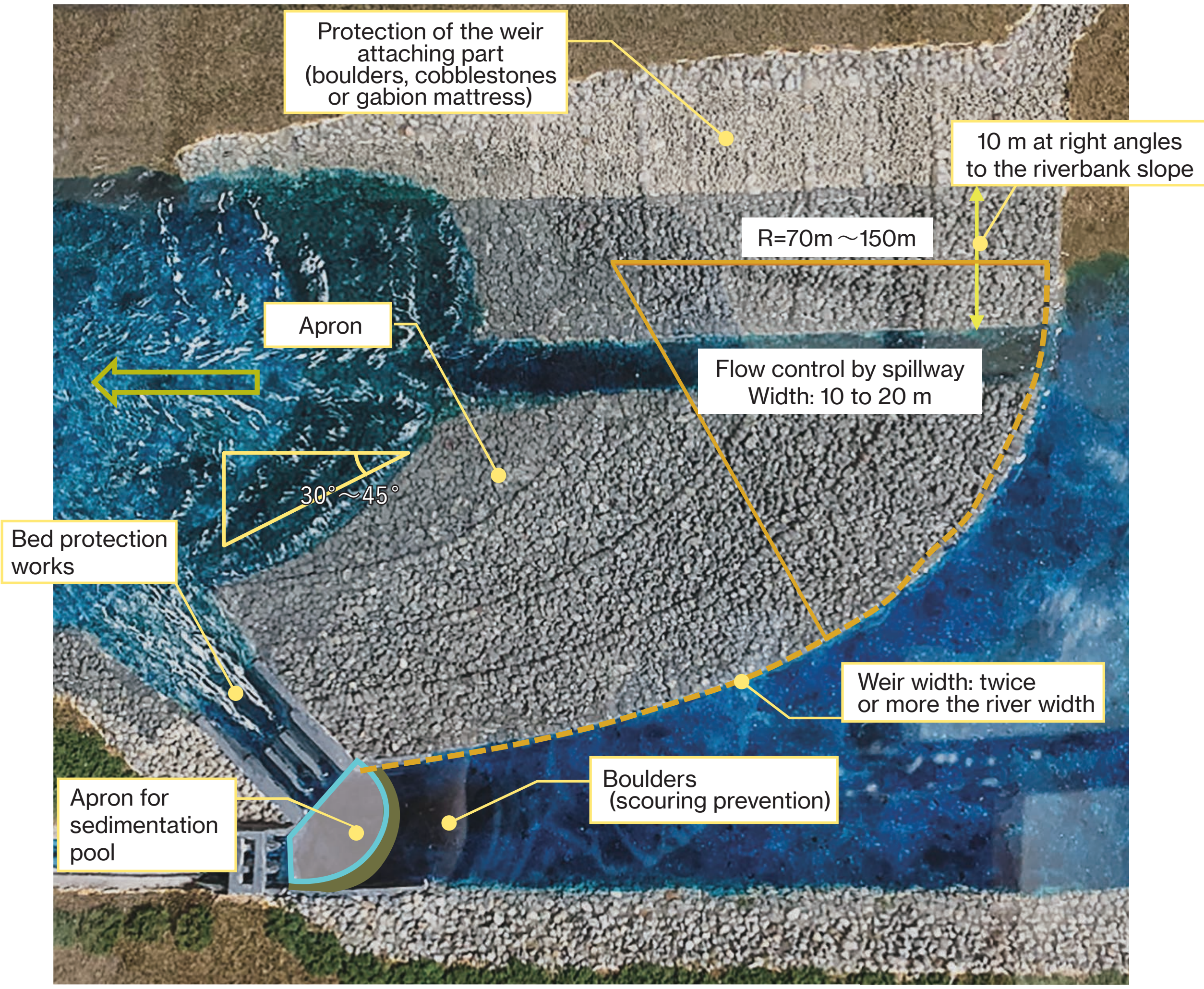


Figure 4.10 Plane Drawing of the Boulder Oblique Weir [2)]

(3) Layout Design of the Intake Gate and Incidental Facilities for the Boulder Oblique Weir

Intake facilities comprise an intake weir and intake gate while a sand flushing ditch, spillway and weir wings protection are also provided in the boulder oblique weir as incidental facilities. The layout of these intake gate and incidental facilities is as follows:

- The intake gate will be located on the riverbank connecting to the boulder oblique weir abutment. The weir crest functions as a training wall conveying the river water into the intake gate.
- The sand flushing ditch is provided on a part of the weir body adjacent to the intake gate located on the riverbank connecting to the boulder oblique weir abutment, discharging soils accumulated upstream of the intake weir to prevent sediments from flowing into the intake gate. Moreover, a flush board of the sand flushing ditch is provided at an abnormally low water level to secure the water level of the intake gate. Accordingly, the sand flushing ditch also serves as a movable weir.
- A spillway is provided at around the top of the convex-shaped boulder oblique weir gathering flows of the spillway and sand flushing ditch into the center of the river channel to set off their force. The spillway is 10 to 20 m width and at least 1 m depth. The flow of the sand flushing ditch is directed to the river center.
- To control the intake amount at the intake gate in the case of abnormal drought and discharge sediments at the sand flushing ditch, movable flush boards shall be provided at the gate pier. Accordingly, the gate pier and foundations of the intake gate and sand flushing ditch are constructed by reinforced concrete to firmly fix them in place.

- A reinforced concrete apron is provided as a sedimentation pool immediately upstream the intake gate and sand flushing ditch to form a solid integrated structure of the intake gate, sand flushing ditch and apron. The apron gradient is set on the side of the sand flushing ditch to promote sand flushing.
- Consecutive protection works 10 m width or more and 50 m length (weir length) are provided to ensure the weir wings in the opposite bank of the intake gate are attached to the river banks and sand bars.

(4) The place where Hydraulic Jump Occurs

As shown in Figure 4.11, the water overflowing-weir channels to the apron from several directions at the curved weir crest. It is deemed that when each water momentum is triggered (decelerated), a hydraulic jump occurs at around the midpoint of the apron. Since apron works also function to protect the bed originally provided immediately after the hydraulic jump point, a curvature radius allowing overflowing water from multiple directions to gather at the middle of the apron must be set when deciding on the curved shape (radius) of the oblique weir crest. Specifically, the curvature radius is set to ensure the center of the radius of arc in the major overflowing part is within the area of boulders filled in the apron. Accordingly, the area of boulders filled in the apron covers where hydraulic jumps occur in the apron.

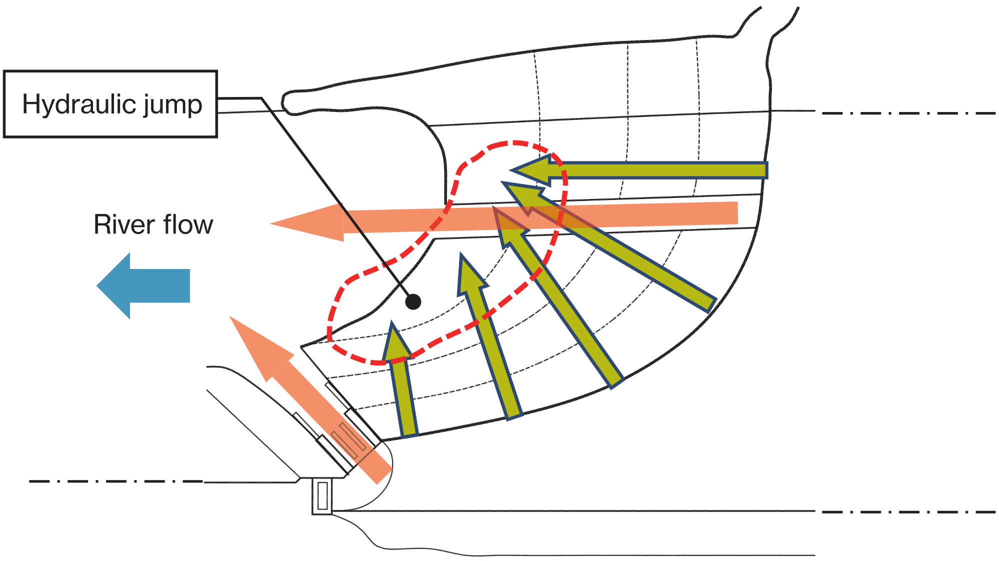


Figure 4.11 Hydraulic Jump Location at the Weir Apron [2)]

Photo 4.3 shows the photographs before and after the rise of river water level during the transition from the drought season to the flood season at the intake weir location where the above mentioned phenomenon can be observed. Between late April and early May when the water level rose, hydraulic jumps in the apron more or less occurred in the apron area filled with boulders.

Kama Weir I viewed from downstream: February/25/2019

Marwarid Weir II viewed from upstream: January/3/2018

Kama Weir I viewed from downstream: April/26/2019

Marwarid Weir II viewed from upstream: May/5/2019

Photo 4.3 Aerial Views of the Oblique Weir [1)]

4.2.4 | Basic Specification Design for Intake Weir and Intake Gate

The first basic specifications to be determined when designing the intake weir and intake gate are the base elevation of intake gate, weir crest elevation and weir overflow level (see Figure 4.12). The following basic specifications adopted in past PMS method irrigation projects can be referenced. However, as a general rule, the base elevation of intake gate is set to a design drought water level based on the lowest level during a winter season (see Chapter 3) and shall remain unchanged until further notice.

- Base elevation of intake weir (as a general rule, the design drought water level): 0.7 to 1.0 m higher than the current riverbed height and 0.5 to 0.8 m lower than the weir height
- Intake weir height (weir crest elevation): 1.2 to 1.8 m higher than the current riverbed elevation
- Overflow water depth at the intake weir (the overflow level is the design intake level of the intake gate): 0.5 to 1.5 m

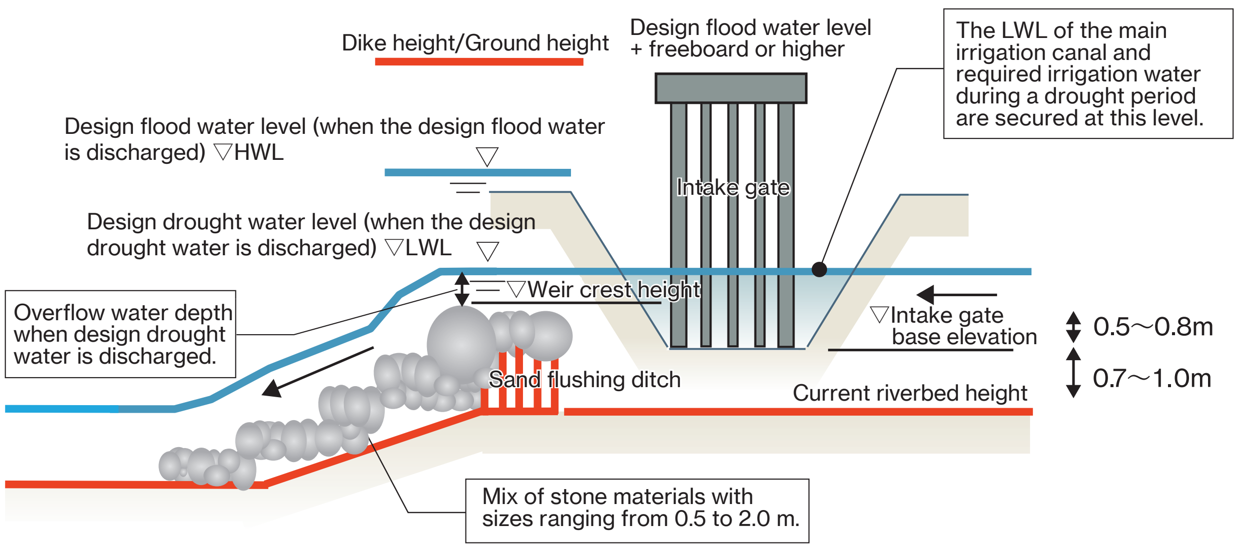


Figure 4.12 Design Specifications for Intake Weir/Gate Elevation Setting [2)]

These basic specifications for intake weir and intake gate are determined by the following process of trial and error based on the elevation difference with the irrigation beneficiary area and the design water level of the standard cross-section of main irrigation canal (see Figure 4.13):

1) The design drought water level of the river is determined as the intake gate base elevation.
2) The weir height is assumed to be between 1.2 to 1.8 m, referencing the existing PMS irrigation project.
3) The design drought (design intake) water level at the intake gate is determined as the overflow level when design drought water is discharged in the weir. The overflow water depth is calculated by the following overflow formula as reference:

Overflow formula $Q = CBH\sqrt{2gH}$ ·· (4.2) [2), see 5)]

Here, Q: overflow amount, C: overflow coefficient (around 0.35 when completely overflowing), B: overflow length, H: overflow water depth and g: gravitational acceleration (g=9.81)

4) Based on the intake gate base elevation, design drought intake water level and elevation of irrigation beneficiary area as determined, the gradient, standard cross-section and design water level of the main irrigation canal are assumed by which the required irrigation water can be discharged.
5) Assuming that the main irrigation canal is constructed with the gradient and standard cross-section as above, the need for land acquisition is considered and the workability and construction costs analyzed by calculating the quantity of works, including embankment and excavation, to consider the validity of the gradient and standard cross-section assumed.
6) In case if they are insufficient, the weir height and overflow level are reconsidered reverting to process 2). For example, when making the canal width narrower than the assumed standard cross-section, the design drought water level at the intake gate is increased by raising the weir height or increasing the overflow water depth.
7) Combination of basic specifications and gradient and standard cross-section of main irrigation canal is considered continuously until reaching their optimum balance.
8) When finding the optimum combination, their values are determined eventually as the basic specifications for intake weir and intake gate and gradients and the standard cross-section of the main irrigation canal.

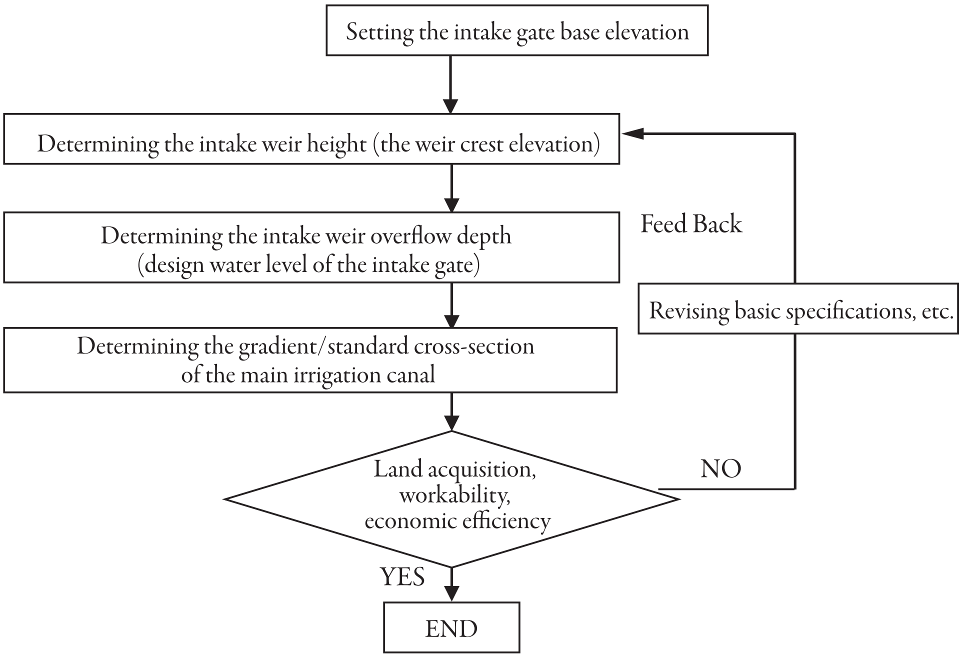


Figure 4.13 Process of Setting Basic Specifications for Intake Weir and Intake Gate [2)]

4.2.5 | Specification Design of the Intake Weir

(1) Cross-sectional Design of the Boulder Oblique Weir

Following the determination of basic specifications for intake weir and intake gate (intake gate base elevation, weir crest elevation and weir overflow level), the cross-section of the boulder oblique weir is designed as follows (see Figure 4.14):

- A spillway is provided around 20 m - or at least 10 m - from the opposite bank where the location separates the wings from the main flow during a flood. Its bottom width is 10 to 20 m in accordance with the aforementioned plane design. The spillway base elevation is around the intake gate base elevation (reference height) at the weir crest, ensuring that the river water can be conveyed into the intake gate side, even during the severest of droughts.
- The reference base level of the sand flushing ditch is −0.7 to −1.0m and it is 2.0 m x 4 spans width. The span width of the sand flushing ditch is standardized by the flush board length, which is, in turn, determined in terms of yield strength against sand and water pressures on the board. The number of spans for the sand flushing ditch depends on the design drought water discharge of the river, which is determined by the method described in "4.2.7 (5) Cross-sectional Design of Sand Flushing Ditch (span width and the number of gates)".
- Based on the above concepts, designs (spans and height) for the intake gate and sand flushing ditch are standardized and the number of spans arranged according to the flow amount in each basin. To do this, the flush boards are also standardized.

If the design intake level is not achieved due to riverbed deformation or drought once the irrigation facility operation is underway after completion of works, sufficient irrigation water shall be secured by decreasing the cross-sectional size of the spillway or increasing the span numbers of the intake gate.

In preparing a cross-sectional drawing of the intake weir, the following shall be noted:

- As well as specifying external dimensions and specifications, such as width, base elevation and crest height, for intake weir and intake gate, sand flushing ditch and spillway, the basic conditions for the water level, including the design drought water level, design flood water level, design intake water level and overflow

level are specified.
- Basic specifications are set out for the concrete, boulders gabion and other materials comprising each structure.
- The consistency of each structure shown on the cross-sectional drawing shall be in line with its location on the plane drawing.

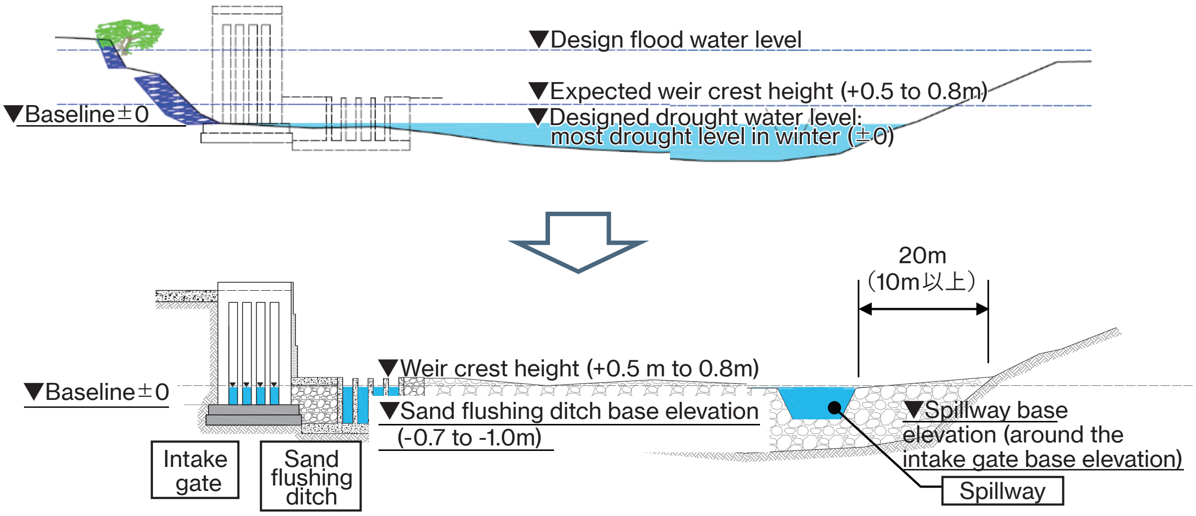


Figure 4.14 Cross-sectional Drawing of the Boulder Oblique Weir [2)]

(2) Profile Design for the Boulder Oblique Weir

The profile design for the boulder oblique weir is prepared as follows:

- The weir length is sufficiently secured and the weir body crest inclined downstream to increase the height of the entire river channel by piling boulders.
- The longitudinal gradient downstream of the weir body including the apron shall be about 1/70 to 1/50 (1.5% to 2.0%) so that the stability of the boulder oblique weir can be ensured. Thanks to this arrangement, the structure prevents scouring by setting off the tractive force with the immediately downstream flow from the weir crest during flooding.
- The weir body foundation shall secure an embedment 1 m deep or more (penetrating depth from the riverbed) from the current deepest riverbed on both the upstream and downstream sides. At the same time, boulders are also filled in on the current riverbed surface for around 20m to the river profile in the area furthest downstream of the apron, to prevent scouring between the weir body and current riverbed ground surface.
- The apron on the upstream side of the weir body is usually provided to prevent scouring of the riverbed by overflow. For an oblique weir constructed by stone masonry, it is considered that the upstream part of the weir body is protected by piling up boulders at an internal friction angle in water (around 38° when using cobblestones which is the maximum slope angle capable of maintaining stability voluntarily without collapsing when they are piled up in water), functioning as the upstream apron.

The length of the boulder oblique weir includes the weir body and apron, which can be calculated by the gradients of the riverbed around the weir location, weir body and apron. If the apron is located at a steep slope, the overflow velocity increases and may impair the weir body stability. If lowering the apron gradient, particularly where the riverbed gradient is steep, the weir length becomes longer and the cross-section is rendered economically inefficient. Accordingly, the profile gradient of the intake weir is set within the scope of its stability by "4.2.5 (4) Confirmation of the Stability of Stone Materials composing Boulder Oblique Weir" ensuring that an excessively low gradient shall not be set. Figure 4.15 shows an image for the consideration of weir length.

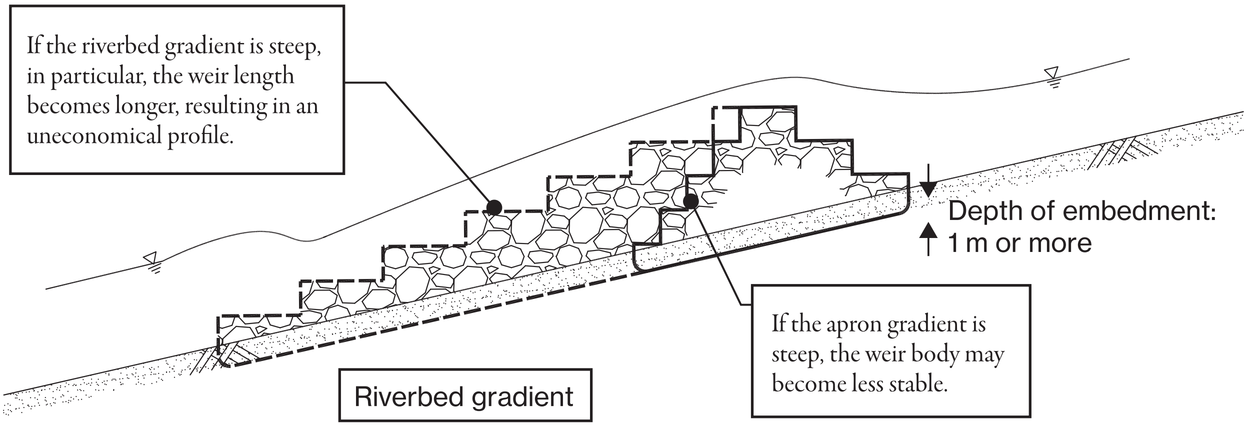


Figure 4.15 Profile Design of the Boulder Oblique Weir: an image for considering weir length [2)]

(3) Protection of the Boulder Oblique Weir Part Attaching to Riverbanks and Sand Bars

As shown in Figure 4.16, when a solid weir body constructed by boulders is placed on soft gravel and soil, gaps between them are scoured, resulting in significant erosion. Accordingly, the parts to which the intake weir wing and river banks/sand bars are attached shall be as long as possible to ensure stability of the weir body wings.

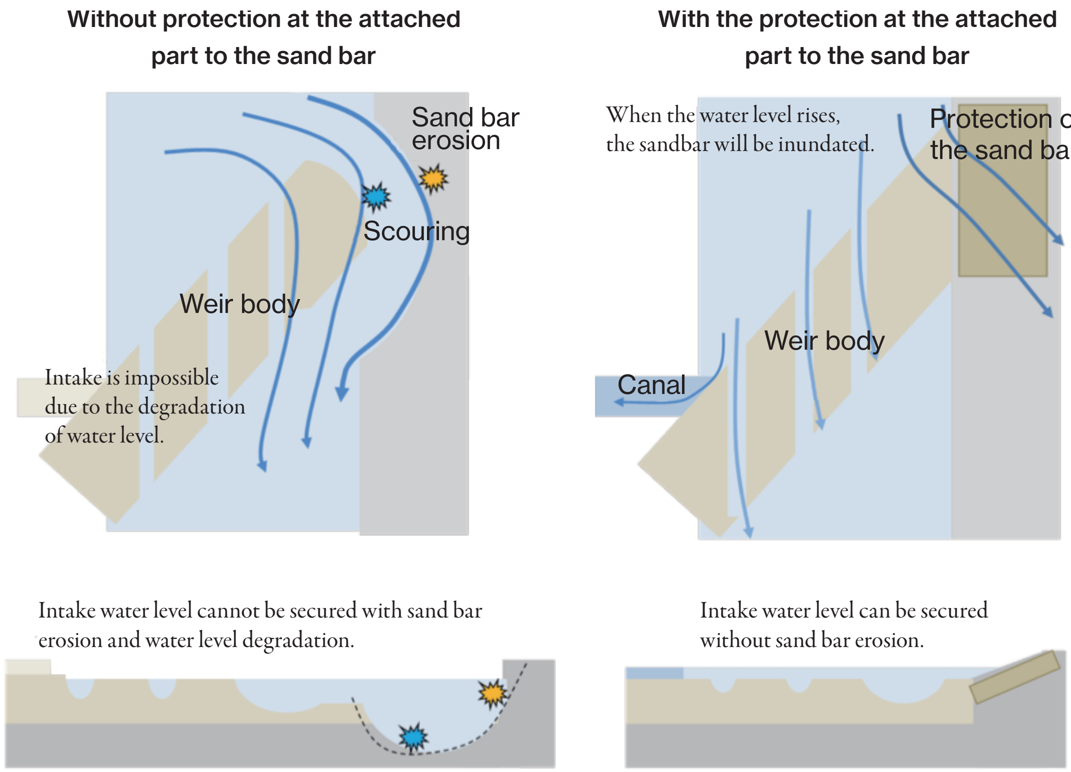


Figure 4.16 Scouring of the Boulder Oblique Weir Parts attached to River Banks/Sand Bars [3)]

To protect all the sand bars supporting the intake weir body wings, "gabion grid connection" or "needle-like fascine works" methods applying fascine works in gabion works shall be applied, as shown in Photo 4.5 and Figure 4.17. However, considering workability and applicability to rivers with high flow velocity like the Kunar River in mind, "cobblestone filling in the boulder frame method" is established as the ultimate form of sand bar protection as shown in Figure 4.18, whereby all sides are surrounded by boulders and cobblestones are filled

inside.
Although the early PMS method irrigation project adopted sand bar protection using a gabion as shown in Photo 4.4, it is not applied nowadays to protect sandbars. However, there are still cases where only gabions are applied for the purpose of preventing soil from being drawn out at the edge of the sand bar in some cases.

Photo 4.4 Foot Protection Works in the Sand Bar (Kama Weir I) [1)]

Photo 4.5 Gabion Grid Connection in the Foot Protection Works (Kama Weir II)) [1)]

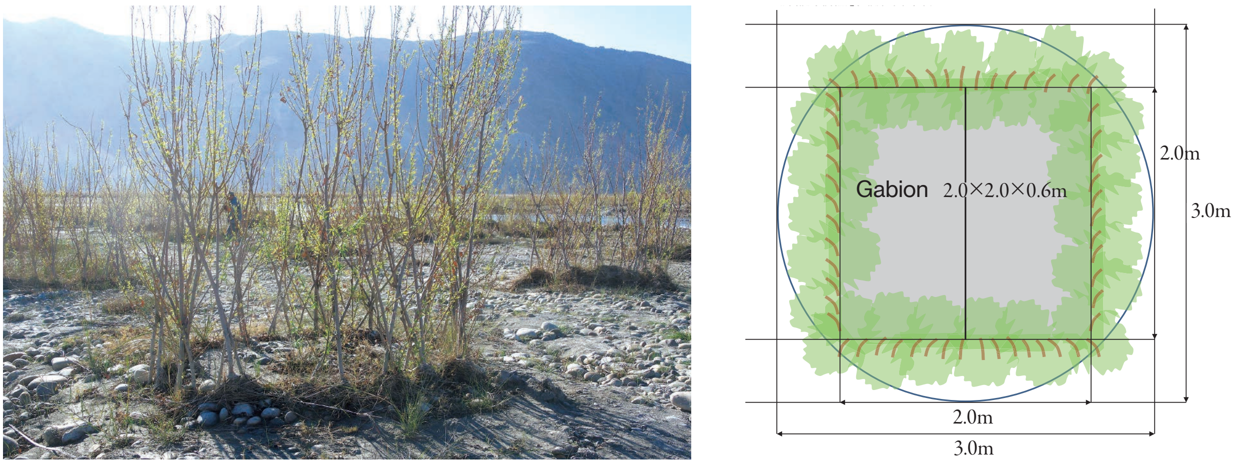


Figure 4.17 Example of "Needle-like Fascine Works" [3)]

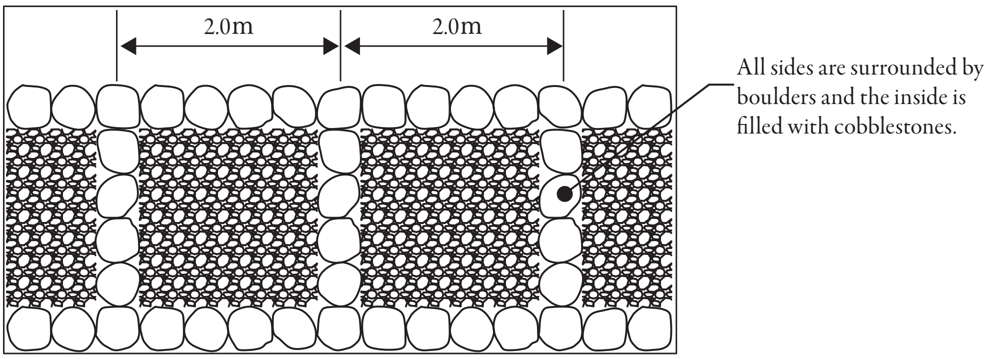


Figure 4.18 Example of a Method of "Cobblestones filled inside the Boulder Frame" [1)]

(4) Confirmation of the Stability of Stone Materials Comprising the Boulder Oblique Weir

In a narrowing river channel and with the weir body and apron at a steep slope, flow accelerates and its impact on the weir body intensifies. Since the boulder oblique weir body comprises masonry and boulders, the stability of the latter needs to be confirmed. In other words, there is a need to construct the weir body using larger boulders that do not move and are not eroded, even when the design flood water is discharged over the weir.

A stone size comprising a stable boulder oblique weir shall be set as it ensures stability against the tractive force of higher flow velocity among 1) critical flow at the weir crest, 2) flow velocity at the apron and 3) flow velocity at the time of overflow (the area downstream of the weir apron). If boulders of a designated size are unavailable around the project site, there is scope to design a stable weir with available stone materials by lowering the gradient of the weir body and apron, securing the overflow weir width to reduce the unit width discharge of the design flood water discharge or taking other measures.

When considering the velocity and depth of the river flow traversing the weir body and apron, 1) design flood water discharge, 2) weir width, 3) weir height and apron gradient (weir length) and other relevant data are required. The image of the river flow is shown in Figure 4.19, and the specific calculation method is described as follows:

a) Calculation of critical flow at the weir crest

The critical water depth and flow around the weir crest portion can be calculated using the following formula. If a unit width discharge can be reduced, the velocity of water overflowing the weir can be reduced.

$$v_c = \sqrt{gh_c}\ ,\ h_c = \frac{q}{v_c} = \frac{q^{\frac{2}{3}}}{g^{\frac{1}{3}}} \quad \cdots\cdots \quad (4.3)$$ [2), see 6)]

Here, v_c: critical flow (m/s), h_c: critical water depth (m), g: gravitational acceleration (m/s^2), q: unit width discharge (m^3/s/m) (q = design flood water discharge / weir width)

Figure 4.19 shows the water surface profile when the river flow goes from a gentle to a steep slope, then reverts back to the channel at a gentle gradient. The flow becomes torrential at a critical level from the normal flow, then reverts to normal after a hydraulic jump. During flow shift from normal to torrential, the critical flow occurring marks the boundary point between normal and torrential flows. The flow velocity and water depth at this moment collectively constitute the critical flow and critical water velocity. The phenomenon at this boundary point can be shown as $F_r = 1$ using the following Froude number (F_r):

$$\text{Froude number: } F_r = \frac{v}{\sqrt{gh}} \quad \cdots\cdots \quad (4.4)$$ [2), see 6)]

Here, v: average flow velocity (m), g: gravitational acceleration (m/s^2), h: water depth (m)

Flow exceeding the critical water depth ($F_r = 1$) is a normal flow ($F_r < 1$) while a shallower flow is torrential ($F_r > 1$). The normal flow is gentle and involves water table fluctuation being transmitted from downstream to upstream. The torrential flow involves great force and water table fluctuation that is not transmitted from downstream to upstream.

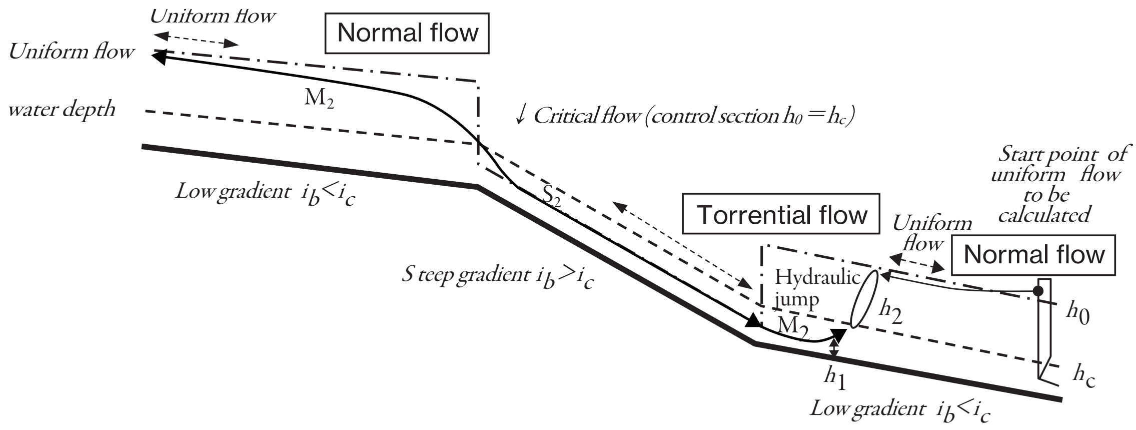


Figure 4.19 River Flow : Normal Flow, Critical Flow, Torrential Flow, Hydraulic Jump and Returning to the Normal Flow 2)

b) Calculation of flow velocity at the weir apron portion

The flow velocity of the weir apron can be calculated using the following Manning formula. The greater the unit width discharge and the steeper the weir apron gradient, the higher the flow velocity.

$$v = \frac{1}{n} R^{\frac{2}{3}} I^{\frac{1}{2}} \quad \cdots\cdots (4.5)\ 2),\ (\text{see } 6))$$

Here, v: average flow velocity (m), n: roughness length, R(=A/S): hydraulic radius (m), A: discharge area (m²), S: wetted perimeter (m), I: riverbed gradient

c) Calculation of flow velocity at the area immediately downstream of the weir apron

i. Design water discharge used for the stability analysis of boulders composing the weir body is calculated by the following formulas (4.6)(4.7) to judge whether the water overflowing the weir is a complete or submerged overflow. In the case of complete overflow with the deigned flood water discharge, the water flow is set as the designed water discharge. In the case of submerged overflow with the designed flood water discharge, the maximum water flow to become a complete overflow (the river flow meeting h_c + D = h_2) is set by the following formulas (4.6)(4.7) as the designed water discharge.

$$\begin{cases} h_c + D > h_2 & \cdots\cdots (4.6)\ ^{2),\ \text{see } 6)} \\ h_2 = \left(\dfrac{Q}{1/n \cdot I^{1/2}} \right)^{3/5} & \cdots\cdots (4.7)\ 2),\ (\text{see } 6)) \end{cases}$$

Here, hc: critical water depth (m), D: drop height (m), h_2: uniform flow water depth downstream (m), Q: water discharge (m³/s), n: roughness coefficient of the river channel, I: riverbed gradient of the channel

ii. Calculation of the flow velocity in the area downstream of the apron (v_{1a})

The flow velocity in the area immediately downstream of the weir apron is calculated by the following formula of energy conservation, while the water depth at the edge of the downstream part of the apron (h_{1a}) is calculated using the following formula and taking frictional damage of the apron slope surface into consideration. The greater the water flow and the greater the drop between the area upstream and downstream of the weir, the higher the flow velocity.

$$\varphi = Z_1 + h_1 + \frac{Q^2}{2gA_1^{\ 2}} - \frac{n_1^{\ 2} l Q^2}{2R_1^{4/3} A_1^{\ 2}} \cdots\cdots \quad (4.8)$$ [2), see 6)]

$$\psi = h_{1a} + \frac{Q^2}{2gA_{1a}^{\ 2}} + \frac{n_{1a}^{\ 2} l Q^2}{2R_{1a}^{\ 4/3} A_{1a}^{\ 2}} \cdots\cdots \quad (4.9)$$ [2), see 6)]

To achieve $\varphi = \psi$ in the above two formulas, the water depth at the edge of the downstream portion of the apron (h_{1a}) is calculated from the critical water depth at the weir crest and the flow velocity (v_{1a}) can be calculated using the following formula (4.10):

$$v_{1a} = \frac{Q}{h_{1a}} \cdots\cdots \quad (4.10)$$ [2), see 6)]

Here, Z_1: weir height, h_1: critical water depth (m), Q: water flow (m^3/s), A_1: flow-section area (m^2), A_{1a}: flow-section area at a low gradient (m^2), n_1: roughness coefficient of the river channel, n_{1a}: roughness coefficient at a low gradient, R_1: hydraulic radius of the channel (m), R_{1a}: hydraulic radius at a low gradient (m), l: weir length (m), h_{1a}: water depth in the downstream of the apron (m), v_{1a}: flow velocity in the downstream of the apron (m/s)

d) Calculation of the stable stone size

The highest flow velocity among the aforementioned critical flow velocity around the weir crest, in the weir apron part and at the overflow water depth (the area immediately downstream of the weir apron) is applied as design flow velocity to calculate the required stone size corresponding to the design flow velocity using the following formula of "stability examination model for stone masonry with low integrity". When the weir apron gradient is low and the Froude number is , F_r no torrential flow, critical water depth or critical flow velocity apply. In this case, the required stone size is calculated using the flow velocity of normal flow in the weir apron part.

e) Stability examination model for stone masonry with low integrity

Earth and water pressures will be major causes of destruction when the slope gradient exceeds 1:1.5 while the fluid force will be a major destruction cause when the gradient is lower than 1:1.5. Since the boulder oblique weir comprising stone masonry using natural boulders has a low gradient, the stone masonry is destroyed by the tractive force, rather than the yield strength exerted from the rear by earth pressure. Accordingly, the stability of the boulder oblique weir shall be considered using the following "stability examination model for stone masonry with low integrity".

For stone masonry with low integrity with adjacent members, the key is whether the critical tractive force of all stone material exceeds the tractive force of the river and remains stable.

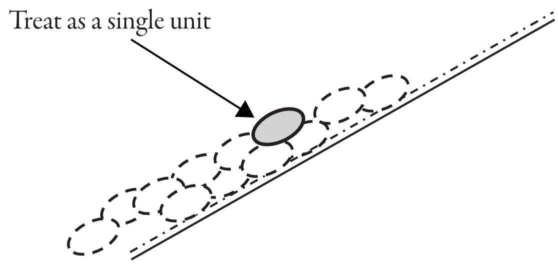


Figure 4.20 The Stability Examination Model for Stone Masonry with Low Integrity in which the Tractive Force of the River Causes Destruction[2), see 7)]

The stability of stone materials used for the boulder oblique weir is examined using the following basic formulas. These formulas are used to calculate the stone size applied in riprap works, determining the relation between the representative flow velocity V_0 and the stone size, since the tractive force exerted on the stone materials does not exceed the movement limit of the stones. Formula (4.11) is for horizontally provided ripraps. When installing ripraps on the slope surface with a slope angle θ , Formula (4.12) is used to calculate the correction coefficient K and the riprap diameter is obtained as $K\ D_m$, multiplying D_m by K.

$$\begin{cases} D_m = \dfrac{1}{E_1^2 \cdot 2_g \left[\dfrac{\rho_s}{\rho_w} - 1\right]} V_0^2 & \cdots\cdots (4.11)^{2), \text{see } 7),8)} \\[2ex] K = \dfrac{1}{\cos\theta \sqrt{1 - \dfrac{\tan^2\theta}{\tan^2\Phi}}} & \cdots\cdots (4.12)^{2), \text{see } 7),8)} \end{cases}$$

Here, D_m: average particle shape of stones (m), V_0: representative flow velocity (m/s), : stone density (kgf･s²/m⁴)【kg/m³】, : gravitational acceleration (m/s²), : water density (kgf･s²/m⁴)【kg/m³】: $\frac{\rho_s}{\rho_w}$ is usually around 2.65, E1: experimental coefficient showing the turbulence intensity (usually, E1=1.2), ϕ : internal friction angle in water of stone materials (around 38° for natural stones while around 41° for crushed stones)

4.2.6 | Specification Design of the Intake Gates

(1) Intake Gate Design Procedures

The intake gates leading to the river water extracted at the intake weir and channelled into the main irrigation canal are designed via the process shown in Figure 4.21.

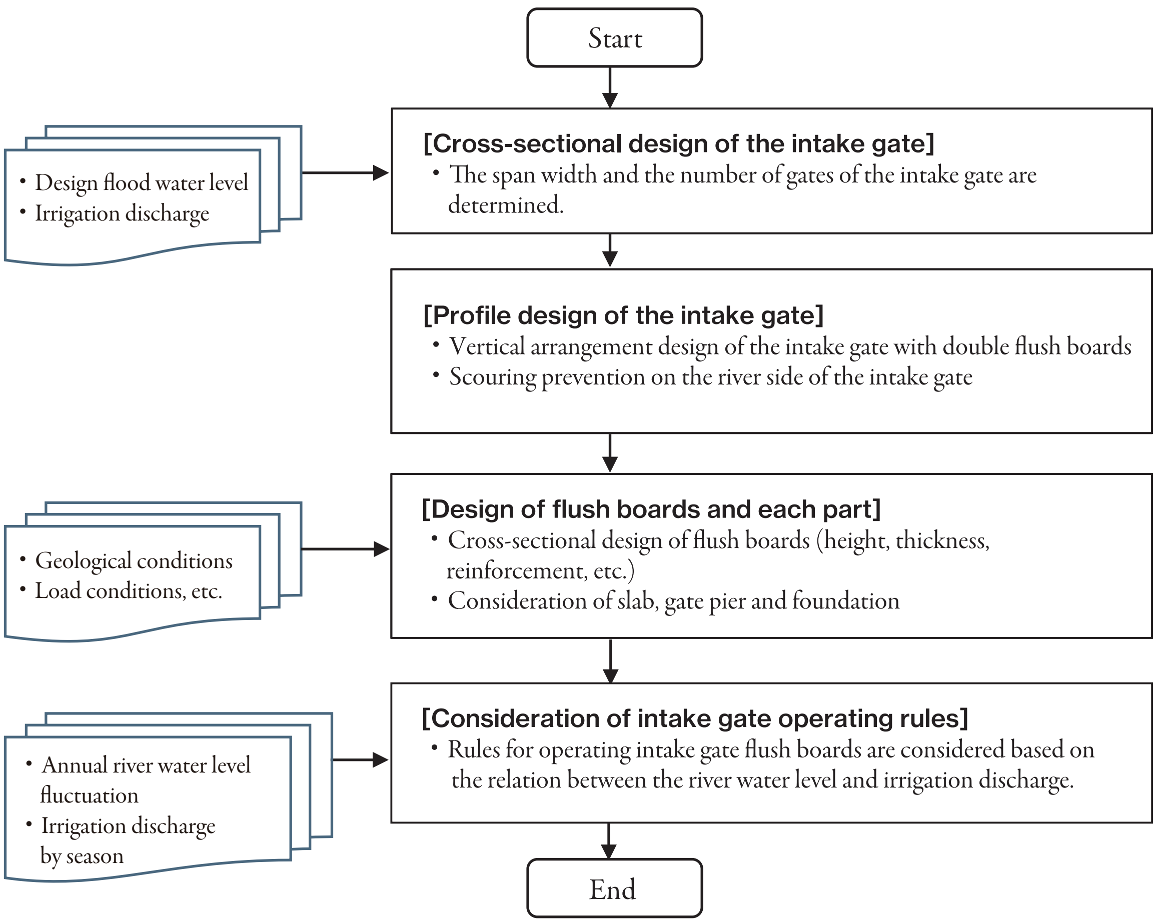


Figure 4.21 Design Process of the Intake Gate 2)

(2) Type of Intake Gate: Double Flush Board Method

Intake gates are connected to the portion of the intake weir wings immediately upstream, comprising double flush board method, gate piers, flush board hoisting equipment, etc. Electrically operated steel gate is adopted in many countries for the gate while the manual double flush board method is adopted in PMS method irrigation facilities. To avoid sand infiltrating the main irrigation canal as much as possible, the overflow water above the flush boards is drawn. A Himalayan cedar flush board reinforced by steel plate is applied, which is hoisted with a Charkha - a traditional equipment in Afghanistan hoisting and lifting a rope in well digging.

Compared with an electrically operated steel gate, the intake gate using the manual double flush board method is affordable when it comes to both initial costs and maintenance and management costs. Since it can be constructed using materials available in Afghanistan (woods, bricks, steel plate, etc.), local residents are fully capable of maintaining and managing the facility. Table 4.3 shows a comparison of the types of intake gate.

Table 4.3 Types of the Intake Gate

Manual double flush boards (adopted as a PMS method irrigation facility)	Electrically operated steel gate (adopted in many countries)
 Source of photo: 1)	 Source of photo: JICA

(3) Basic Policy for Designing an Intake Gate with Double Flush Board Method

According to the result of works in the PMS method irrigation project, the specifications of the double flush board method are shown as follows. As for the span width per intake gate and specifications of flush boards, the existing PMS irrigation project shall constitute the standard benchmark design, taking the yield strength of flush board against the water pressure into account. Meanwhile, the intake gate height, number of gates, interval between the first stage and second stage flush boards and the foundation depth are determined according to their regional conditions, such as design flood discharge, design high water level, required irrigation water and foundation ground in the target region. The items and details for the intake gate design are described as follows. Figure 4.22 to Figure 4.24 show an example design drawing of the double flush board method.

- Cross-sectional design of the intake gate：	The height of the intake gate shall be 3.0 to 4.0 m from the base elevation. The span width and number of gates shall be the width 1.5 m x 4 gates as standard.
- Profile design of the intake gate：	The distance between the first stage and second stage columns of the intake gates shall be secured the interval of 6.5 to 8.0 m.
- Flush board design：	The Himalayan cedar and other Pinaceae boards are applied. Flush board length: 1.7m, height: 20cm, thickness: 5cm, steel plates 4 mm thick are provided on one side of the board as reinforcement. both sides as reinforcement.

- Intake gate structure

The gate pier shall be a reinforced concrete structure.
For the foundation structure and to stabilize the structure, existence of boulder layer is confirmed and cement is poured into the cobblestones and gravel layer (over 50cm) to establish the direct foundation and a reinforced concrete slab (40cm) is provided above the direct foundation.

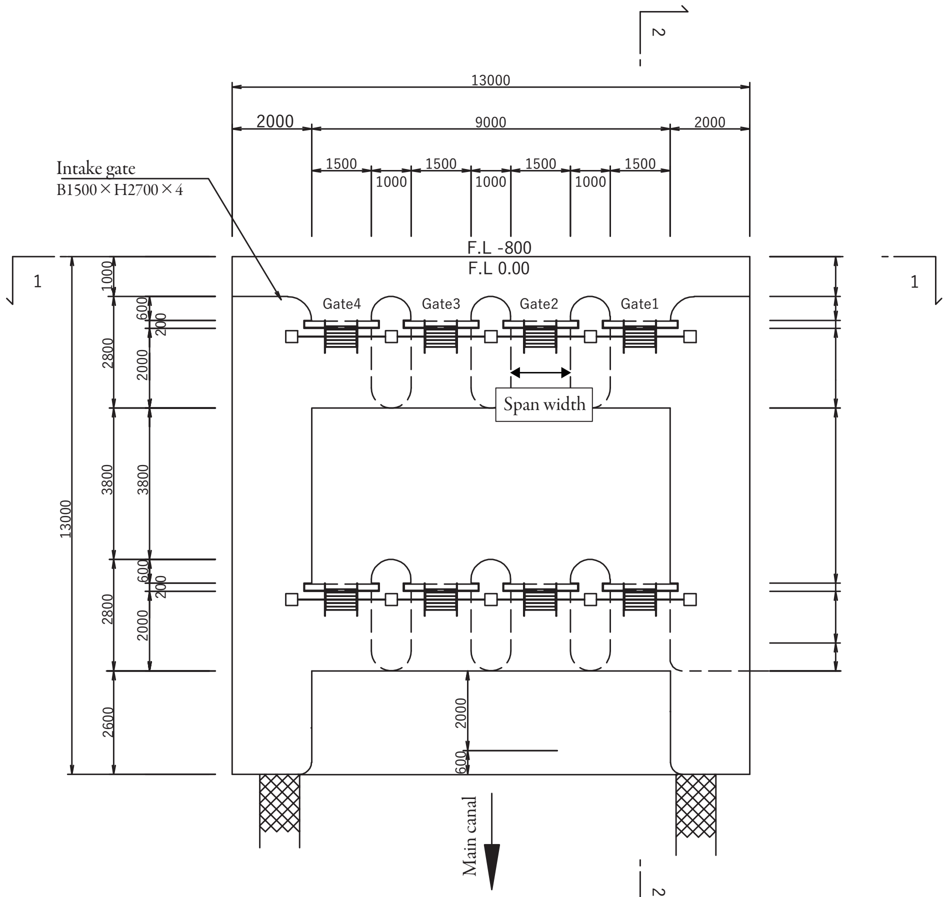


Figure 4.22 Example Plane Drawing of the Double Flush Board Method Intake Gate 3)

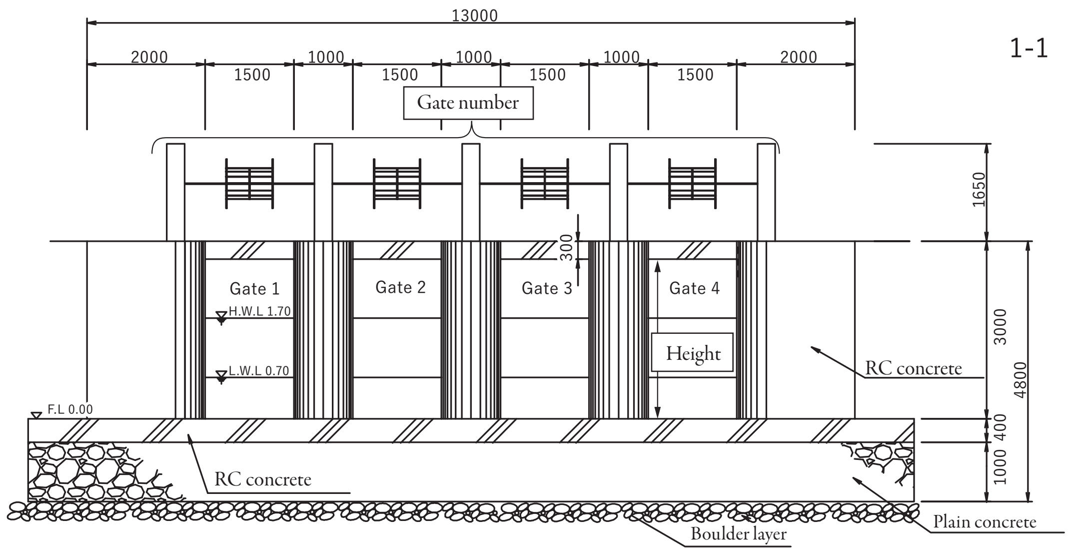


Figure 4.23 Example Cross-sectional Drawing of the Double Flush Board Method Intake Gate 3)

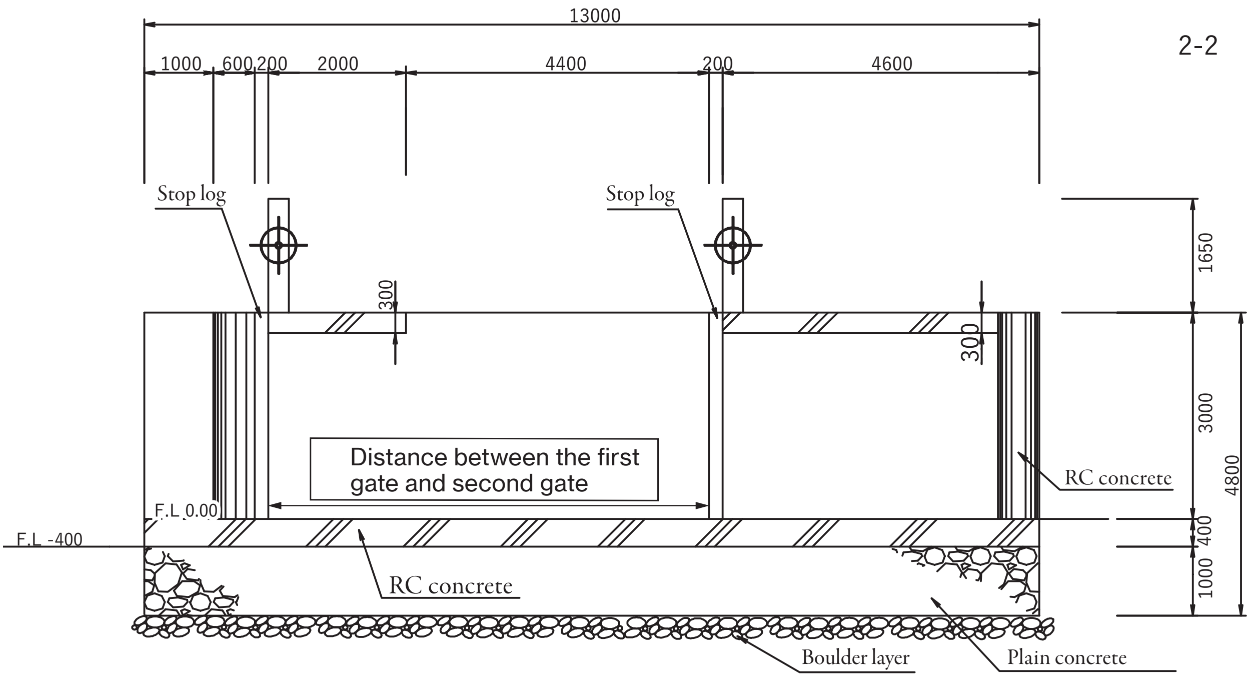


Figure 4.24 Example Profile Diagram of the Double Flush Board Method Intake Gate [3)]

(4) Cross-sectional Design of the Intake Gate (height, span width and the number of gates)

In the PMS method irrigation project, the intake weir is kept lower to minimize flood damage and this explains why the overall width of the intake gate is wider. The standard span width per intake gate shall be 1.5 m considering the yield strength of flush board against the water pressure. This span width is determined by deducting the ditch width of 10 cm on the left and right sides of the standard flush board length of 1.7 m (see 4.2.5 (6) below). The number of spans is determined considering the width of the main irrigation canal as examined in "4.3.3 Standard cross-sectional design of main irrigation canal", The intake gate height shall be secured the height obtained by adding the freeboard shown in Table 5.2 of Chapter 5 to the design flood water level.

(5) Profile Design of the Intake Gate

Controlling the water pressure on the flush board of the first column with the double flush board method as shown in Figure 4.25, the double flush board method of the intake gate causes the water pressure on the first-column board to decline by creating a water pool between the boards. Accordingly, the water pressure generated by the amount of water raised by the second-column board counters the weight on the downstream side of the first-column board, respectively controlling significant water pressure on the lower board portion of the first column. The standard interval between the boards on the first stage and second stage columns shall be 6.5 to 8 m to ensure the function of water level rising by flush board on the second column.

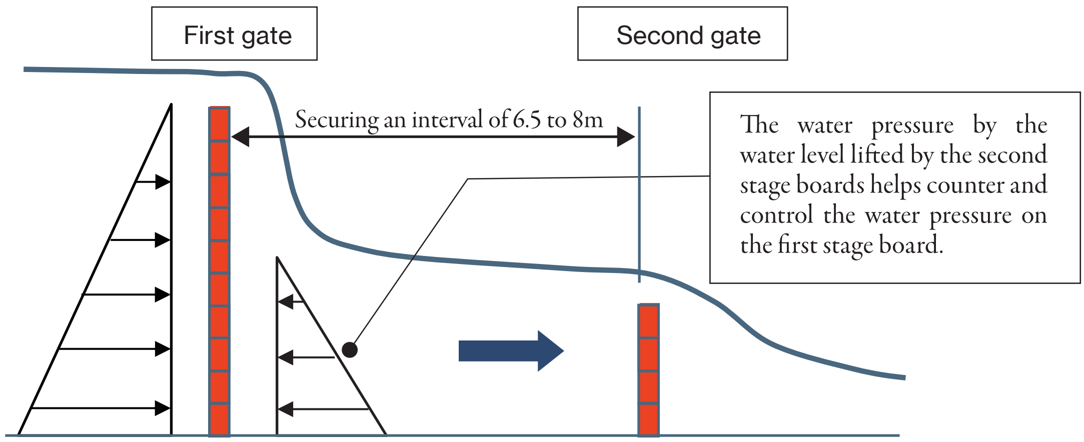


Figure 4.25 Double Flush Board Method Controls the Water Pressure on the First Board 2)

Installation of a sedimentation pool in the area below the immediately upstream of the intake gate
A sedimentation pool of 0.7 to 1.0 m depth is provided below the area immediately upstream of the intake gate to control the sand inflow into the main irrigation canal and promote sand discharge from the sand flushing ditch of the intake weir as shown in Figure 4.26.

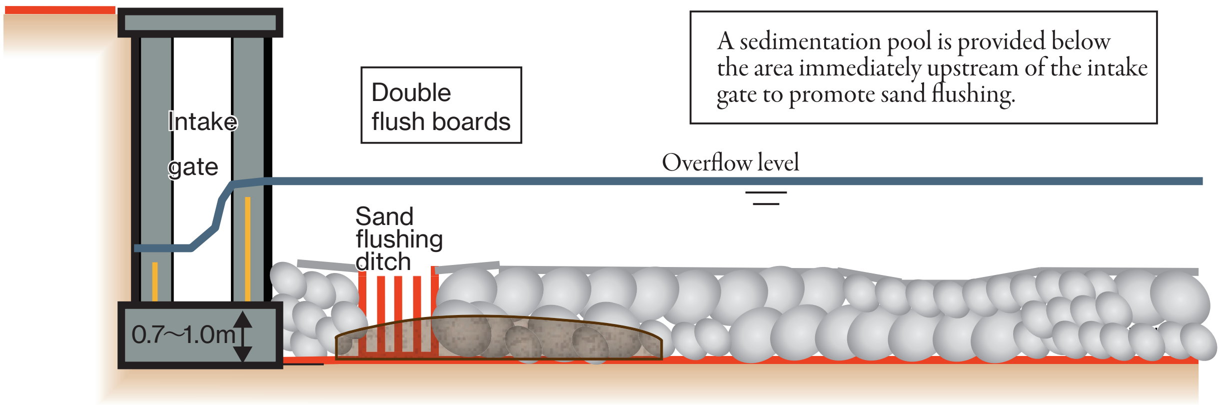


Figure 4.26 Sedimentation Pool Provided below the Area Immediately Upstream of the Intake Gate 2)

(6) Design of the Flush Board

The standard flush board structure applied with the double flush board method shall be 1.7 m length, 20 cm height and 5 cm thickness, with one side of the Himalayan cedar board reinforced by 4 mm-thickness iron plate. This design was obtained as a result of trial and error during the existing PMS irrigation project, examining the yield strength of the board against water pressure, scope to lift the board weight manually using charkha and other factors. Separate examination of strength against bending, shearing and deflection revealed a high level of practicality.

Text Block 4-2: Flush Board Structure with the Double Flush Board Method

The original flush board applied with the double flush board method was only structured by the board 1.7 m length, 20 cm height and 5 cm thickness, which did not work successfully due to buoyancy. Accordingly, a 2.5 mm-thickness iron plate was placed on the board and butyl rubber and rubber sponge were applied to a part attached to another board to ensure sealability. However, these rubber materials easily peeled off, exacerbating the leakage. Eventually, the board was shaved in a straightforward manner at the processing factory stage to ensure sealability between the boards. This method was then successfully applied to intake facilities in Sheiwa and Kama. (a report from Dr. Nakamura: received on April 7, 2010)

Photo 5cm-thickness iron plate is put in place and a hook is welded. A hook at the end of the rope is hooked and the board is lifted by a hoisting device. [1)]

Photo Charkha hoists the board, which lifts up boulders when digging a well. This is traditional engineering of Afghanistan. [1)]

(7) Design of the Gate Pier

As shown in Figure 4.22, the cross-sectional shape of the gate pier provided with the double flush board method shall be an oval with a semi-circular shape on the upstream and downstream sides to reduce flow water resistance as much as possible. The gate pier height shall be at the design flood water level, adding a freeboard or higher. When the size of rectangular cross-section of the gate pier is based on a 1.5 m span width, an empirical formula (4.13) usually shows values as shown in Table 4.4.

Table 4.4 General Relations between the Height and Thickness of the Gate Pier when the Span Width is 1.5m [2)]

Gate pier height	Gate pier thickness
2.0m	0.53m
2.5m	0.59m
3.0m	0.65m
3.5m	0.71m
4.0m	0.77m

An empirical formula: $tp=0.12\,(Dp+0.2Bt) \pm 0.25$ ………………………………… (4.13) [2), see 9)]

Here, tp: gate pier thickness (m), Dp: gate pier height (m), Bt: span width (m)

Referring to the general cross-sectional shapes of the gate pier as above, the gate pier stability is examined based on the structural stability calculation. The following points are confirmed regarding gate pier stability for direct foundations : 1) stability against falling down (rotation), 2) stability against sliding, 3) stability of ground-bearing forces and 4) stress of each member within allowable stress intensity. The gate pier stability in the case of pile foundations needs to be examined separately. The types of load on the gate pier are: (i) the weight of the gate pier, (ii) weight of the flush board hoisting machine, (iii) weight of flush board, (iv) weight of floor slab for the crest part, (v) water pressure on flush board, (vi) earth pressure by sedimentation, (vii) seismic force, (viii) uplift pressure and others.

(8) Design of the Intake Gate Foundation

In most past PMS method irrigation projects, the intake gate foundation was constructed in the gravel layer, mixing and consolidating locally collected aggregates (cobblestones and sands) and establishing the direct foundation at a depth of the 1 m. Further, a solid reinforced concrete slab 40 cm thickness was established on the direct foundation. See Photo 4.6. Although gravel mixed with cobblestones originally has a strong soil bearing capacity even when unprocessed, more rigid ground was formed by said method to secure the stability of the superstructure of intake gate.

Photo 4.6 Intake Gate under the Construction: constructing a reinforced concrete foundation[1)]

Moreover, boulders are filled in the boundary between the sediment pooling apron in the area immediately upstream of the sand flushing ditch and the current riverbed, which prevents scouring of sediment pooling apron foundation.

4.2.7 | Specification Design of the Sand Flushing Ditch

(1) Process for Designing the Sand Flushing Ditch

Figure 4.27 shows the process for designing the sand flushing ditch.

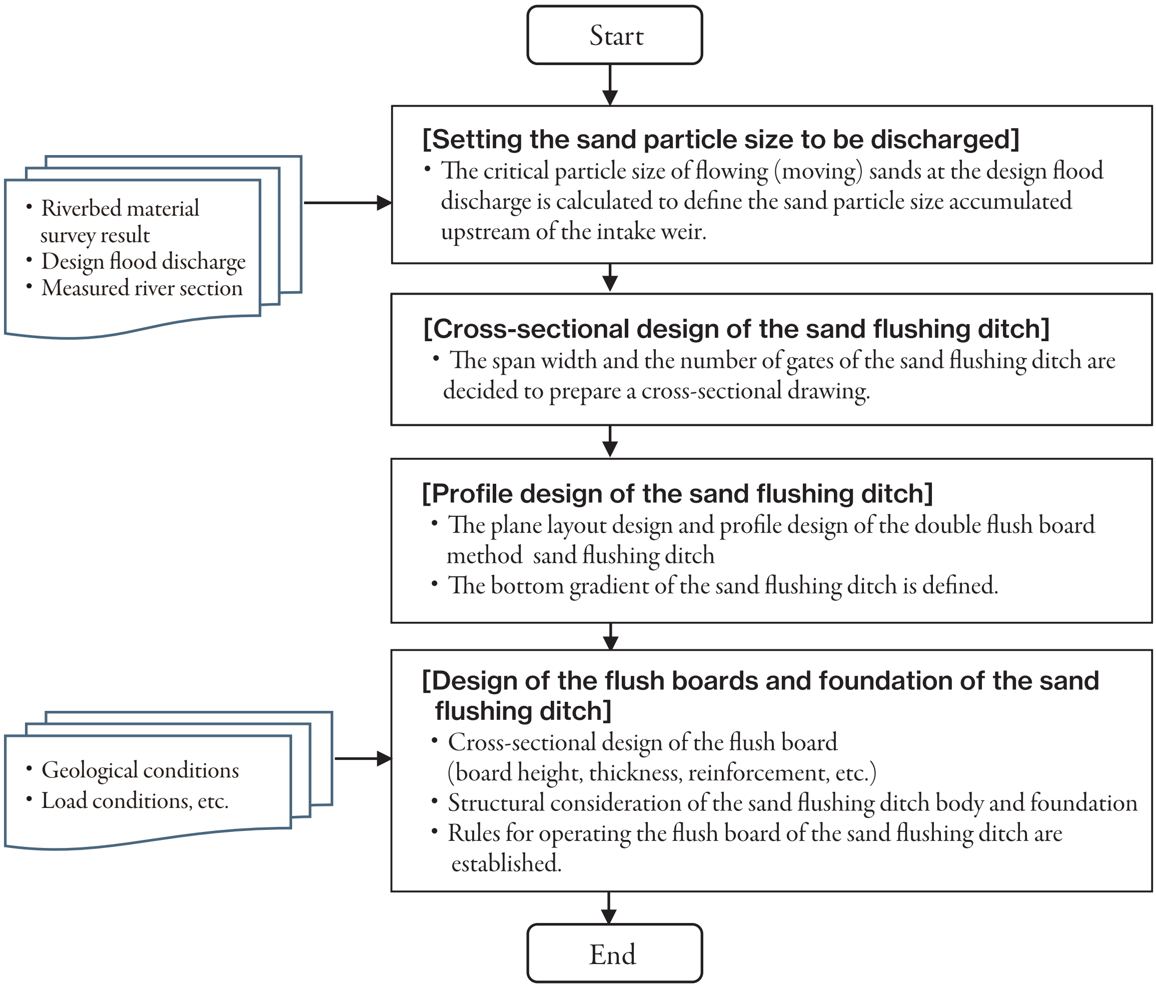


Figure 4.27 Process for Designing Sand Flushing Ditch [2)]

(2) Sand Flushing Method: Sand Flushing Ditch containing "Partially Movable Weir" leveraging Manual Double Flush Boards

In the PMS method irrigation project, a sand flushing ditch is installed as a piece of ancillary equipment for the intake weir with the following functions: 1) discharging bedload flowing toward the intake gate to the downstream side of the weir to reduce the sedimentation below the intake gate and sediment flowing into the intake gate, and 2) functioning as a partially movable weir which secures the intake water level by closing the sand flushing ditch when drought water levels are abnormally low. In other words, the sand flushing ditch of the intake weir in the PMS method irrigation project shall also function as a "partially movable weir". This ditch is installed by cutting off part of the intake weir constructed near the intake gate, comprising a reinforced concrete channel, gate pier, double flush boards and hoisting equipment. The board thickness, height and material specifications shall be the same as those of the flush boards applied in the double flush board method intake gate. The flush board operation involves the use of a Charkha - a traditional tool in Afghanistan used for hoisting a rope when digging a well. To fully perform its function as a partially movable weir in the PMS method irrigation project, the sand flushing ditch also serving as a “partially movable weir” by manual double flush boards shall be standardized equipment that facilitates maintenance and management.

Setting up and maintaining and managing the movable weir with manual double flush boards is more affordable than for a movable weir with an electrically operated steel gate. Since it can be constructed using materials (wood, iron plate, concrete, etc.) easily available in Afghanistan, it is also easier for local residents to maintain and manage it. In addition, when the weir is repaired, it can be temporarily bridged and used as a transportation route. Table 4.5 shows a comparison of the types of sand flushing ditch.

Table 4.5 Types of Sand Flushing Ditch

Partially movable weir by manual double flush boards (applied as a PMS method irrigation facility)	Partially movable weir by electrically operated steel gate (adopted in many countries)
 Source of photo: 1)	 Source of photo: JICA

(3) Basic Policy for Designing the Sand Flushing Ditch

According to the result of works in the PMS method irrigation project, the specifications of the sand flushing ditch are shown as follows. As for the specifications of the span width per sand flushing ditch and flush boards, and the structure of the sand flushing ditch body and foundation, those of the existing PMS irrigation project shall be the standard design. The flush board specifications are defined taking the water and earth pressures on the board into account while the structure of the sand flushing ditch between multiple reinforced concrete spans is devised to discharge sediments having accumulated immediately below the intake gate. Meanwhile, the sand flushing ditch height, number of gates and bottom gradient, etc. are determined according to their regional conditions, such as the design flood discharge, water level and riverbed materials while referring to the following specifications from past projects. The items and details for the sand flushing ditch design are described as follows. Figure 4.28 to Figure 4.30 show example design drawings of the sand flushing ditch.

- Cross-sectional design of the sand flushing ditch：	The top elevation of the sand flushing ditch shall be 10 to 20 cm lower than the intake weir crest. The bottom elevation of the sand flushing ditch shall be 0.7 to 1.0 m lower than the bottom elevation of the intake gate. The span width and the number of gates shall be the width 2.0 m x 4 gates as standard.
- Profile design of the sand flushing ditch：	The bottom gradient of the sand flushing ditch shall be 5 to 8% (1/200 to 1/125).
- Flush board design：	The Himalayan cedar and other Pinaceae board are applied. Flush board 2.2 m length, 20cm height, 5 cm thickness. Steel plates 4 mm thickness are installed on one side as reinforcement.

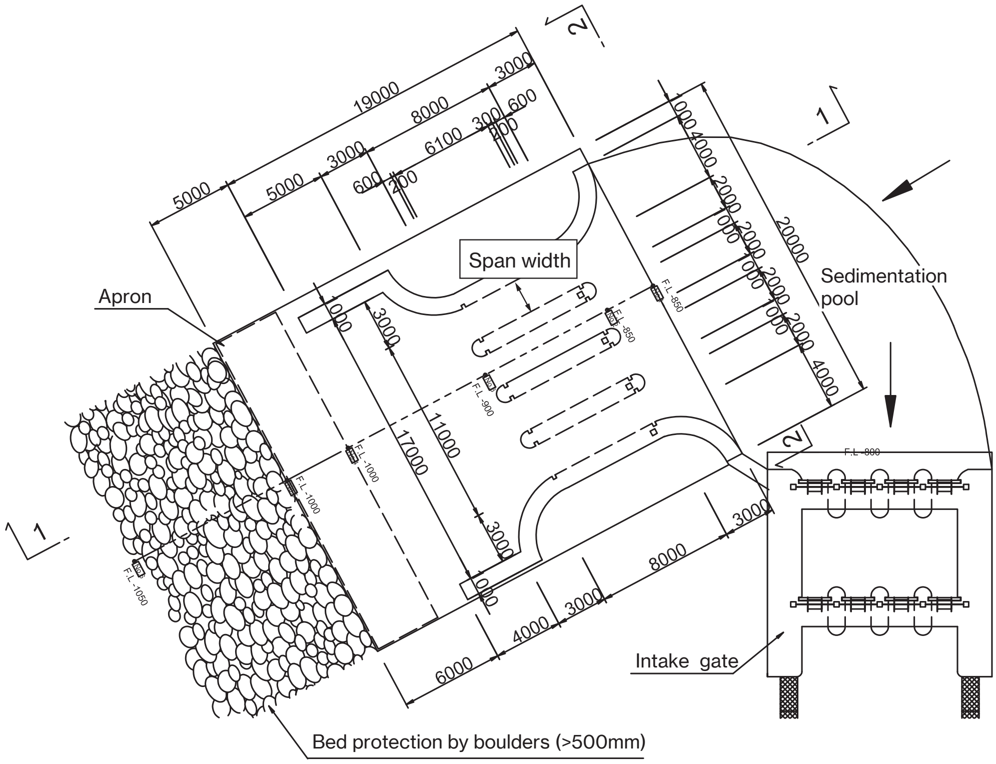


Figure 4.28 Example Plane Drawing of the Sand Flushing Ditch 3)

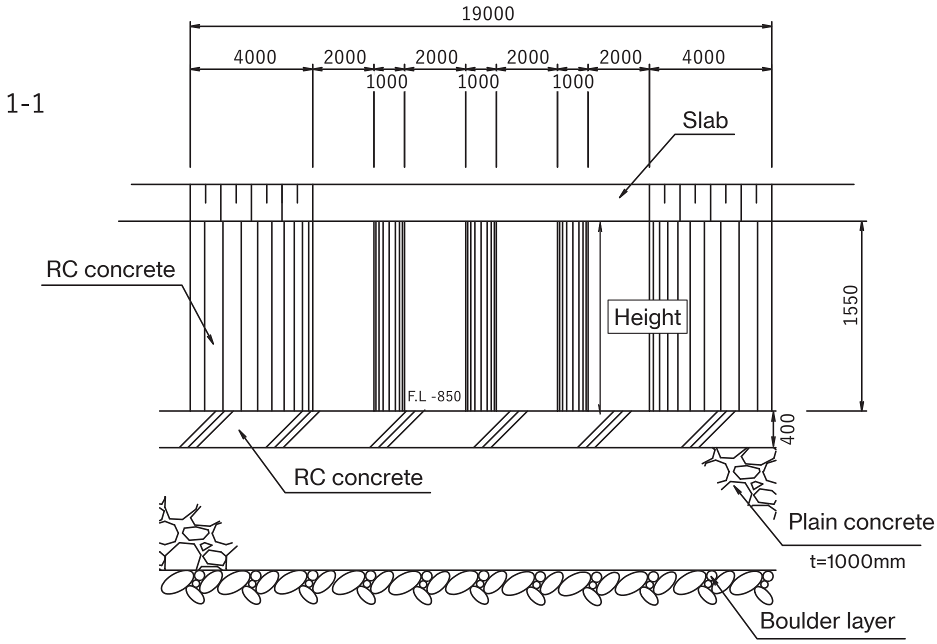


Figure 4.29 Example Cross-sectional Drawing of the Sand Flushing Ditch 3)

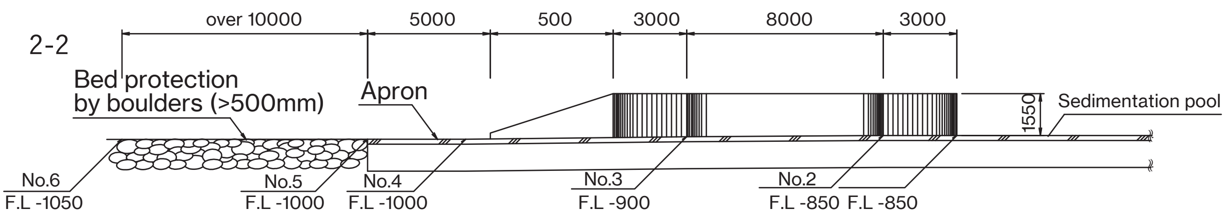


Figure 4.30 Example Profile Drawing of the Sand Flushing Ditch 3)

- Sand flushing ditch structure： The sand flushing channel and gate pier shall be a reinforced concrete structure.
For the foundation structure and to stabilize the structure, existence of boulder layer is confirmed and cement is poured into the cobblestones and gravel layer (over 50cm) to establish the direct foundation and a reinforced concrete slab (40cm) is provided above the direct foundation.

(4) Setting the Particle Size of the Sands to be Flushed

Firstly, the flow velocity V and hydraulic radius R at the designed flood discharge in the target river are calculated using the Manning equation (4.14) to determine the critical friction velocity U_{*c} using the formula (4.15). Using this value allows the particle size of sands to be flushed at the ditch to be calculated as the critical particle size for sediment movement from Iwagaki's formula as shown in Chapter 3. It is expected that gravel of size smaller than the critical particle size flows from upstream during flooding and is accumulated upstream of the intake weir.

Manning equation : $V = \frac{1}{n} R^{\frac{2}{3}} I^{\frac{1}{2}}$ ························ (4.14) [2), see 11)]

Formula for the critical particle size for sediment movement : $U_{*c} = \sqrt{gRI}$ ··············· (4.15) [2), see 11)]

Iwagaki's formula (see Chapter 3): the critical particle size for sediment movement is determined by an empirical formula concerning the relation between the critical friction velocity and critical particle size for sediment movement.

Here: V: flow velocity (m/s), R: hydraulic radius (m), g: gravitational acceleration (m/s^2), I: riverbed gradient, n: roughness coefficient, U_{*c} : critical friction velocity

(5) Cross-sectional Design of the Sand Flushing Ditch (span width and the number of gates)

The span width of the sand flushing ditch shall allow flush boards to withstand the water and earth pressures from upstream and flush away any accumulated gravel. The span width design is basically the same design as the double flush board method intake gate. In terms of dimensions, the flush board is 1.7 m length, 20 cm height and 5 cm thickness with a span width of 1.5 m for the intake gate (see 4.3.5 (4)) while the sand flushing ditch is 2.2 m length, 20 cm height and 5 cm thickness with a span width of 2.0 m. The span width of the sand flushing ditch is secured to exceed that of the intake gate. This is because it may become difficult to achieve sufficient flow velocity to flush away accumulated gravel, since the smaller span width at the sand flushing ditch will cause the hydraulic radius to decline: R=(A/S) (A: flow section, S: wetted perimeter) resulting in a smaller flow velocity and causing the flow velocity determined by the Manning equation to decline. As the height of the sand flushing ditch (around 1.5 m) is lower than the intake gate (3.5 to 4.0 m), the flush board of the ditch is subject to less water and earth pressure than the intake gate, hence the span can be widened.

Further, the base elevation of the ditch bottom has the same height consecutively with the sedimentation pool apron in the area immediately upstream of the ditch and is almost equivalent in height to the current riverbed.

Sufficient sand flushing gates are secured to ensure the flushing volume at the ditch, even at the designed drought water discharge in the target river. The target flow rate in the sand flushing ditch is determined by the "designed drought water discharge - flow rate in the spillway" while the number of flushing gates are determined by the "target flow rate in the ditch ÷ flow rate per gate".

The flow rate per gate can be calculated from the cross-section and vertical gradient (as described later) of the sand flushing ditch using the Manning equation. Photo 4.7 shows the sand flushing ditch with double flush boards.

Photo 4.7 Sand Flushing leveraging the Double Flush Boards 1)

(6) Profile Design of the Sand Flushing Ditch

Since the edge of the sand flushing ditch is prone to riverbed scouring due to the harsh flow of water and sand, sufficient bed protection works shall be secured over several dozen meters. As a minimum, the bed protection works for the sand flushing ditch shall be installed until the tip of the apron in the area downstream of the intake weir body.

The vertical gradient of the bottom of the sand flushing ditch shall allow sand of the target particle size to be flushed as considered in the above (4). With the formula as shown in the above (4), the critical friction velocity is determined from the hydraulic radius (R) and vertical gradient (I) at the sand flushing ditch and the critical particle size for sediment movement in the ditch is calculated using Iwagaki's formula. Subsequently, the vertical gradient (I) is defined using the Manning equation to ensure this critical particle size exceeds that of the target sand particle.

(7) Design of the Gate Pier, Flush Board, and Foundation of the Sand Flushing Ditch

The same design method as shown in "4.2.6 Design of Intake Gate" is applied for the gate pier, flush board and foundation of the sand flushing ditch.

4.3 Design of the Main Irrigation Canal

4.3.1 | Basic Policy for the Design of the Irrigation Main Canal

When designing the main irrigation canal, the priorities are shown as follows. The procedures for designing the main irrigation canal are shown as the design process in Figure 4.31.

- A canal route with good workability and allowing smooth land acquisition is selected for the main irrigation canal. For these routes, rock excavation or routing over farmland or private land shall be avoided as far as possible. While establishing a consensus with regional residents, the route is decided and the construction works shall be promoted.
- When constructing the main irrigation canal on soft ground, the foundation shall be treated carefully, ensuring the main irrigation canal is not damaged by piping occurred on the bed.

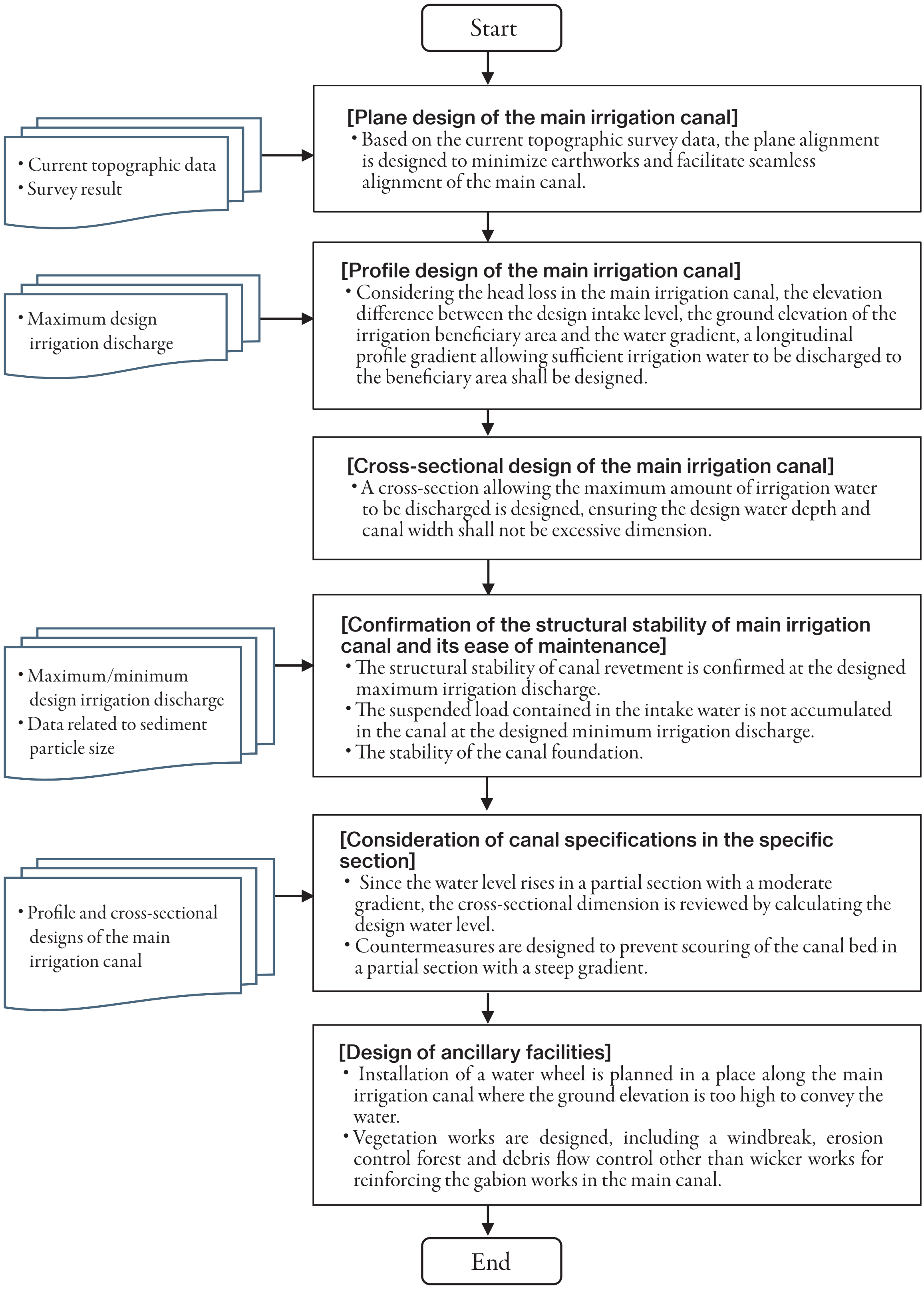


Figure 4.31 Design Process for the Main Irrigation Canal[2)]

4.3.2 | Type and Design Policy for the Main Irrigation Canal

(1) Type of the Main Irrigation Canal

The main irrigation canal comprises the revetment on the canal side and the bottom lining. The canal type with precast concrete lining or unsupported excavation earth canal are usually applied. In the PMS method irrigation project, the standard canal comprises a gabion revetment combined with wicker works and a bottom lined with soil cement.

This canal type in the PMS method irrigation project is affordable compared with a concrete canal. Since the materials used (stones, wire for gabion and cement) are available in Afghanistan, even if it breaks, it is easy to partially repair it and local residents are fully capable of maintaining and managing the facility. Table 4.6 shows a comparison of the types of main irrigation canal.

Table 4.6 Types of the Main Irrigation Canal

The canal with a gabion revetment combined with wicker works and a bottom lined with soil cement (applied as a PMS method irrigation facility)	The canal with concrete lining (adopted in many countries)
 Source of photo: 1)	 Source of photo: JICA

(2) Design Policy for the Main Irrigation Canal

The design specifications of the main irrigation canal in the existing PMS irrigation project are shown as follows. The following structure and roughness coefficient of the canal shall constitute the standard design of PMS method irrigation project. Meanwhile, the route, length, gradient, cross-section and design water depth of the canal are designed in accordance with regional conditions, such as the topographical features and irrigation discharge in the target region while referring to the following actual specifications from past projects. Figure 4.32 show an example design drawing of the main irrigation canal.

- Main irrigation canal structure:	The side bank of the main irrigation canal comprises a gabion and wicker works. The canal bed structure is lined with soil cement.
- Roughness coefficient of the main irrigation canal:	A roughness coefficient of n=0.012 to 0.013 is adopted.
- Main irrigation canal gradient:	Steep gradient main irrigation canal: I=0.001 to 0.0015(1/1000 to 1/670) Main irrigation canal: I=0.0006 to 0.001(1/1,670 to 1/1,000)
- Main irrigation canal section:	Main irrigation canal bed width: 4.0 to 5.0 m width Main irrigation canal depth: 1.6 to 2.0 m depth
- Design water depth of the main irrigation canal : (Design water discharge)	At the minimum irrigation discharge (LWL) 0.4 to 0.6 m (2.5 to 6.0 m^3/s) At the maximum irrigation discharge (HWL) 0.7 to 0.9 m (5.5 to 11.0 m^3/s)

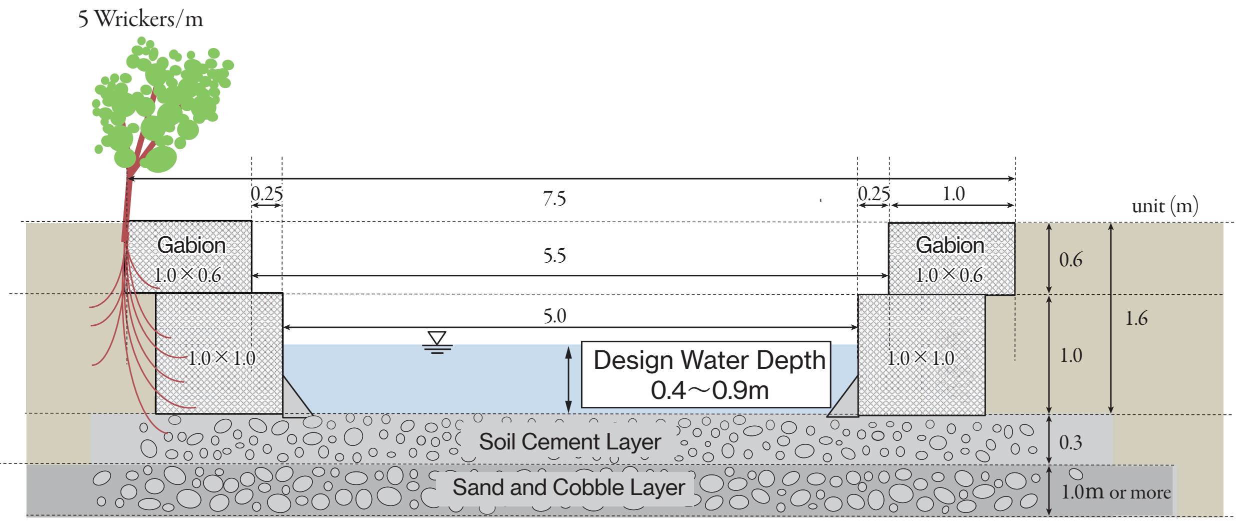


Figure 4.32 Example Cross-sectional Drawing of the Main Irrigation Canal [2), see 3)]

4.3.3 | Specification Design of the Main Irrigation Canal

(1) Plane Design of the Main Irrigation Canal

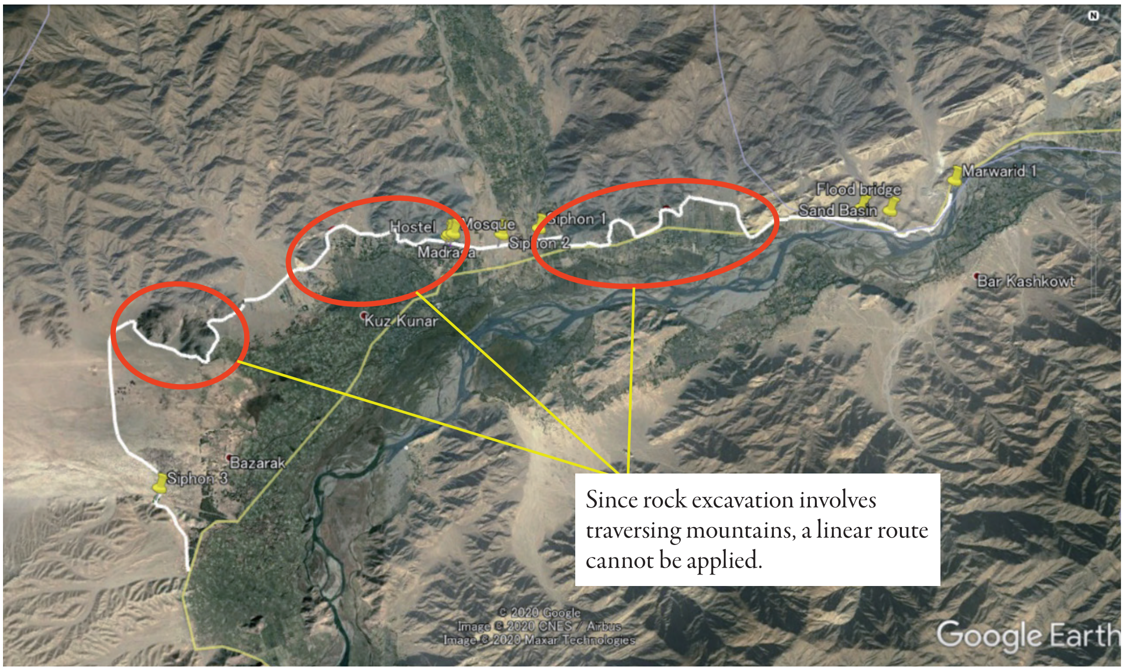
The plane design procedures for the main irrigation canal are shown as follows:

1) Prior to designing, a topographic survey is conducted at the central line defined in the main irrigation canal, mainly carrying out a profile and cross-sectional survey.
2) The standard main irrigation canal cross-section, as considered in 4.3.3 (3), is inserted in the cross-sectional drawing surveyed along the planned main irrigation canal route. The elevation and plane location of the main irrigation canal are defined from the outlined main irrigation canal route considered in Chapter 2.
3) Lines are inserted in the center and at both ends of the main irrigation canal on the provisional canal cross-sectional drawing and their points are plotted on the plane drawing.
4) The plane alignment of main irrigation canal is determined by joining the points plotted on the plane drawing in smooth linear alignment. Subsequently, a plane drawing of main irrigation canal is prepared, which consists of reviewing the plane alignment of main irrigation canal to ensure the line connecting both ends does not overlap private houses and farmland.
5) The central line of the main irrigation canal shall preferably be straight. When incorporating a curve in the central line of main irrigation canal, the curve radius shall be ten times (or at least five times) larger than the canal width to facilitate the flow condition.
6) As above, the plane alignment of main irrigation canal is confirmed. The slope lines of the center and both ends of the main irrigation canal on the plane drawing are inserted in the surveyed cross-sectional drawing to complete it.

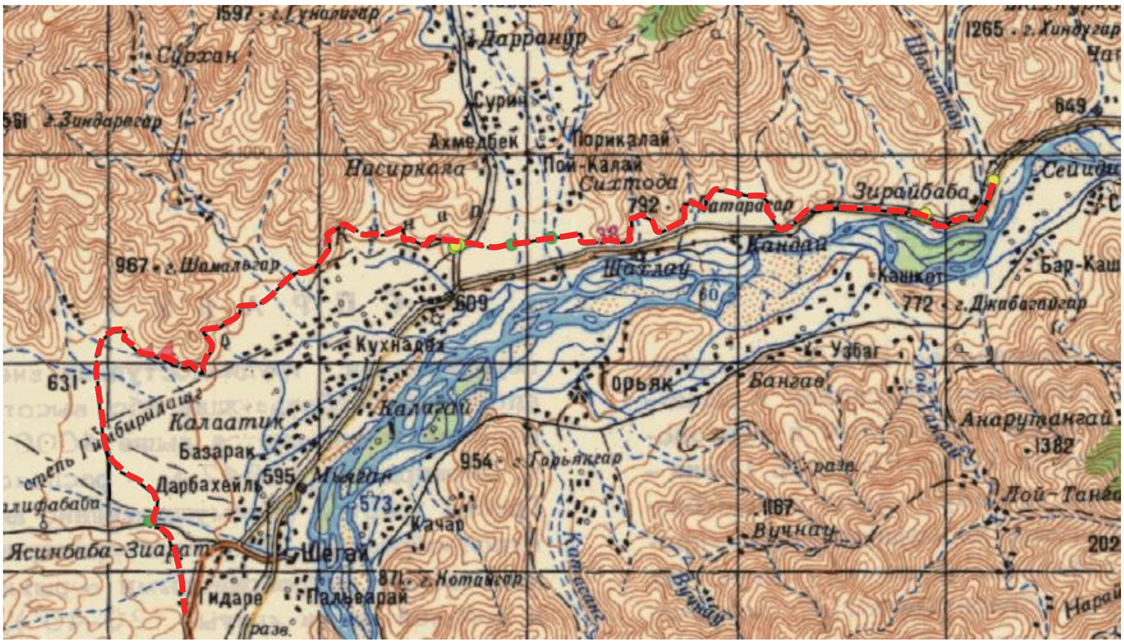
When preparing the plane design of the main irrigation canal, the following points shall be noted:

- Since large-scale excavation and embankment will particularly affect workability, hill and valley terrain shall not be selected as the site. Any route requiring excavation of hard rocks shall also be avoided in particular. In very hilly places or those with many valleys, it is impossible to define a linear route for main irrigation canal. Accordingly, it is preferable to set the route along the contour line, as shown in Figure 4.33.
- The stability of the cut slope and large-scale embankment is fully considered. In particular, a proper slope gradient and small steps shall be considered in the design and stability of the embankment and their construction shall be fully supervised during the works. The design and stabilization measures for large-scale embankments are shown in "4.5.2 Type and Design Policy for the Reservoir, Siphon, and other Facilities" and "4.5.4 Cross-section and Structural Design of the Reservoir ".

- When the main irrigation canal traverses any sections which may be bisected by roads, other canals, rivers, streams, wadi and debris flows, measures shall be taken by applying the canal as a siphon or installing a flood crossing bridge, for example (see Figure 4.54).
- When the main irrigation canal is provided on a river floodplain, dikes, spur dikes and other flood countermeasure works are included in the plan.



Satellite view of the main irrigation canal route in the Marwarid Weir I



Topographic view of the main irrigation canal route in the Marwarid Weir I

Figure 4.33 Setting the Main Irrigation Canal Route along the Contour Line [2)]

(2) Profile Design of the Main Irrigation Canal

The profile design of the main irrigation canal is prepared as follows. And Figure 4.34 and Figure 4.35 show the design examples of the main irrigation canal.

- Regarding the main irrigation canal structure, the revetment on the side bank comprises gabion and wicker works and a soil cement lining for the canal bed. With the lining structure of the canal bed, the Manning roughness coefficient will be n=0.012 to 0.013.
- Based on the intake gate base elevation as determined in 4.2, the profile design of the main irrigation canal is prepared. The longitudinal gradient of the main irrigation canal is linked to its flow velocity. If the gradient is too gentle, sedimentation in the canal progresses. On the other hand, if the gradient is too steep, stability of canal revetment is impaired and the soil cement lining on the canal bed is removed. Accordingly, the reference gradient of the steep gradient main irrigation canal shall be I=0.001 to 0.0015 (1/1000 to 1/670), while that of the other main irrigation canals shall be I=0.0006 to 0.001 (1/1,670 to 1/1,000).
- In the profile design of the steep gradient main irrigation canal, the canal gradient and flow velocity are defined while ensuring no sedimentation occurs in the canal. To ensure that the stability of gabion applied as the canal revetment and soil cement on the canal bed do not deteriorate, the flow velocity of the steep gradient main irrigation canal shall preferably be set below 1.7 to 1.8 m/s.
- Whether sedimentation occurs in the main irrigation canal or not is confirmed as follows: at the minimum irrigation discharge, the critical particle size for sediment movement is calculated using the Manning equation, formula of critical friction velocity and Iwagaki's formula as shown in Chapter 3. When this critical particle size exceeds the sediment particle size included in the canal, it is evaluated that no sediment accumulates in the canal.
- Meanwhile, sediment is reduced in the sand basin, which allows head loss to be controlled by lowering the canal gradient of the main irrigation canal and distributing the irrigation water to a wide area along the canal. However, it is desirable to secure a minimum flow velocity of about 0.7m/s or more in the main irrigation canal.
- As a general rule, the water in the irrigation canal flows at a constant gradient. However, when the elevation difference between the design intake water level at the intake gate and the ground elevation in the irrigation beneficiary area is large, drop works are installed in the main irrigation canal or other measures shall be taken. Conversely, when the above difference is small, low gradient sections are partly installed. There may also be a need to set a steep or low canal gradient locally when there are hard rocks or unavoidable structures along the way. In such cases, the following measures shall be taken, but no low gradient sections are installed in the case of a steep gradient main irrigation canal:
  - Since flow velocity declines and the canal water depth on its immediately upstream section rises in gentle-gradient sections, the uniform flow calculation at the section is conducted to determine the canal water depth and confirm the lack of canal revetment height to ensure the canal cross-section whose revetment is higher than that of the standard section.
  - Since the flow velocity increases in steep gradient sections, raising the risk of partial erosion of the revetment and canal bed, the riverbed is made uneven using stones and bricks to increase roughness and reduce the flow velocity.

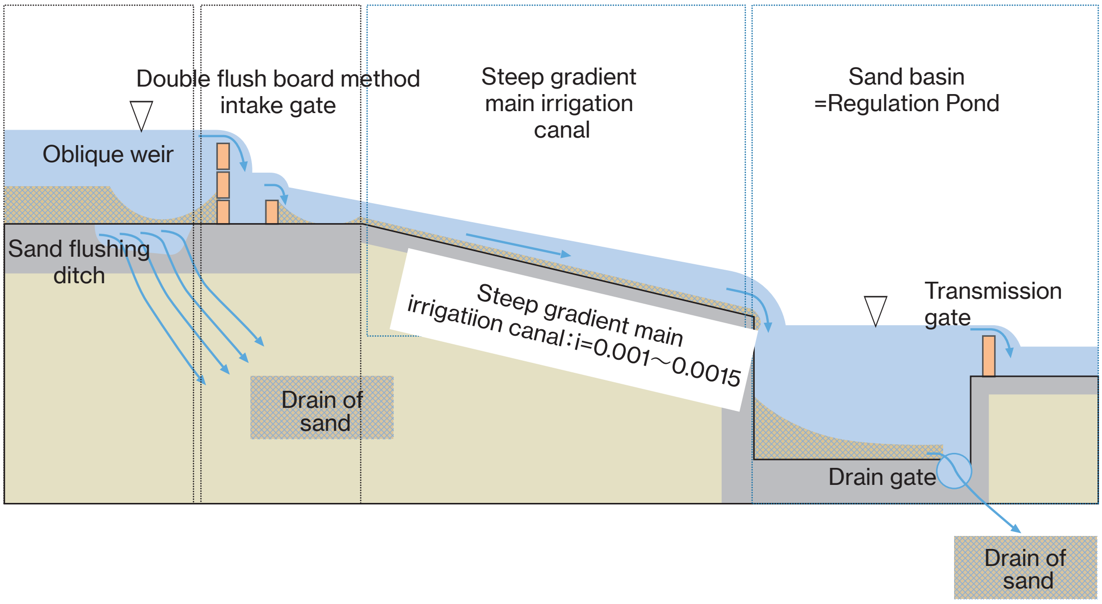
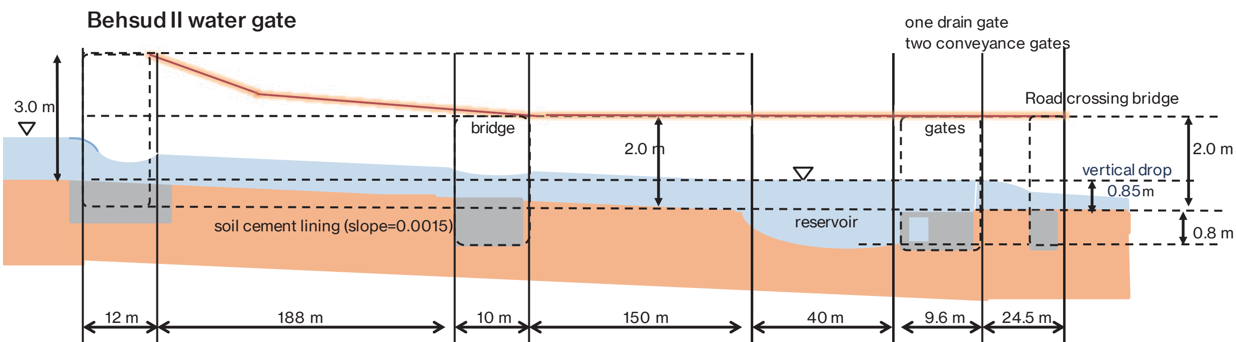


Figure 4.34 Example Profile Drawing of the Steep Gradient Main Irrigation Canal [2), see 3)]



structure	water gate	main canal	bridge	main canal	reservoir	2nd gate	canal joint	total
length	12	188	10	150	40	9.6	24.5	434.1 m
incline	0.0025	0.0015			plain level	0.0063	0.0045	0.0020
vertical drop	0.030	0.527			0.120	0.060	0.110	0.847m

Kunar River at Meeran ; HWL 2.0～2.5 m, LWL 0.2～
(maximum 2.5～3.0 m when the largest flood observed in 2010, 2013)

Figure 4.35 Example Profile Drawing of a Steep Gradient Main Irrigation Canal [1)]

(3) Standard Cross-sectional Design of the Main Irrigation Canal

The standard cross-sectional design of the main irrigation canal is prepared as follows:

- The standard cross-section of main irrigation canal comprises multiple gabions and wicker works in the side bank and an average revetment gradient of 1:0.5; revealing an inverted trapezoid shape with a single cross-section. The canal bed shall be a soil cement lining structure (see Figure 4.32).
- The cross-section of main irrigation canal is designed to allow the required irrigation water to discharge as considered in Chapter 2. The water discharge amount in the main irrigation canal is calculated using the Manning equation.
- In designing the bottom width and depth of the main irrigation canal, the width shall be as wide as possible and the design water depth as low as possible to ensure the heading-up by the intake weir is not excessive. It is preferable to set the cross-section of main irrigation canal to determine a large hydraulic radius (R) by the Mining equation (wetted perimeter(s) is/are shorter). When the relation between water depth (h) and canal width (w) is 1:2, this generally achieves a hydraulically optimum cross-section.

However, the wider the canal width, the more land needs to be acquired and the deeper the water depth in the canal needs to be to increase the weir height. Accordingly, the balance between the canal width, water depth and canal gradient shall be taken into consideration. In terms of revetment stability, the canal depth is set to ensure that the revetment height is 5 m or less. When the height likely exceeds 5 m, cutting the back soil of the revetment or partially rerouting the main irrigation canal shall be considered.

- Since the intake gate base elevation is set at the lowest water level, the bottom height of the subsequent main irrigation canal usually becomes lower than the ground elevation along the main irrigation canal so that the main irrigation canal has an engraved cross-section. In addition, the design water depth of the main irrigation canal is set as low as possible. Accordingly, the main irrigation canal in the PMS method irrigation project allows to secure sufficient freeboard.
  As a reference, the means of calculating freeboard in a general irrigation canal is shown in the following formula:

$$F_b = 0.05d + \beta \cdot h_v + h_w \quad \cdots\cdots (4.16)$$ [2), see 10)]

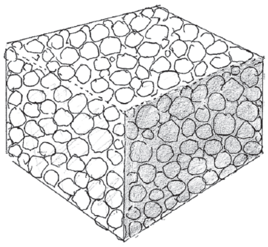
Here, F_b: freeboard height (m), d: water depth to the design water discharge (m), h_v: flow velocity head (m) (=V2/2g), V: flow velocity (m/s), β : conversion coefficient to static head at the flow velocity head (=0.5 to 1.0), h_w: freeboard to the water surface vibration (m) (=0.1 to 0.15m), g: gravitational acceleration (=9.81)

4.3.4 | Structural Design of the Main Irrigation Canal

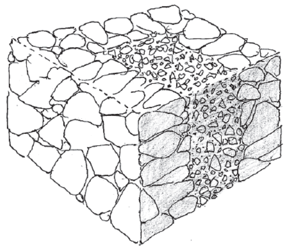
In order to establish a main irrigation canal that is easy and stable for beneficiary farmers to maintain and manage, proper construction materials available on site shall be considered. In the PMS method irrigation project, gabion works using stone materials and annealing wires and wicker works are applied for the revetment of main irrigation canal and soil cement lining is applied to reinforce the canal bed.

(1) Revetment of Main Irrigation Canal (gabion and wicker works)

Gabion works are applied for the internal walls of the main irrigation canal, combining with wicker works. Gabion is unbreakable, flexible, easily repairable, adaptable to vegetation/ecosystem, affordable and has other features. While uniform cobblestones are usually filled in the gabion applied in many countries, large square stones (rubble) are piled up on all sides in a wall-like shape and smaller size gravel is filled to complete this work as shown in Figure 4.36. The case body becomes much more stable, the weight increases more than when uniform cobblestones are filled in and almost no suction occurs in the back. The example of constructed gabion works is shown in Photo 4.8.



Filled by uniform cobblestones (adopted in many countries)



Combination by large square stones and smaller size gravels

Figure 4.36 How to Fill Stone Materials in Gabion Works [3)]

Photo 4.8 Example Gabion Works and Wicker Works in the Main Irrigation Canal [1)]

Annealing wires forming the gabion to be applied have an external frame of 4 mm and a net part of 3 mm, galvanized, soft, handmade, and available in Afghanistan and its neighbouring countries. Each hexagonal side of the mesh shall be twisted three times to ensure it will not come loose easily, even if a single part is torn. Table 4.7 shows the specifications of the gabion. Photo 4.9 shows the production process of the gabion frame, and Photo 4.10 shows the photo of the annealing wires.

Table 4.7 Specifications of Gabion applied in the PMS Method Irrigation Project [3)]

	Height	Width	Length	Mesh diameter	Gabion weight	Capacity	Stones filled	Total unit weight
Gabion A	600㎜	1000㎜	2000㎜	Approx. 120㎜	16kg	1.2㎥	Square stones of over 20 cm diameter or cobble stones of over 15cm diameter	Approx. 2000kg
Gabion B	1000㎜				21kg	2.0㎥		Approx. 3300kg

Photo 4.9 Gabion Production [3]

Photo 4.10 Galvanized Annealing Wires [3]

As for wicker works, 10 to 12 wickers, 40 to 80 cm length and 15 to 20 mm in diameter are planted per 1 m^2 as shown in Photo 4.11. They are densely planted, remain short as shrubs, and widely rooted. Thin roots are divided like a small web and fill the gap between stones in the gabion, fixing the stone materials and making the gabion steady. Photo 4.12 shows the situation 3 months after the planting.

Photo 4.11 Planting Wickers in Plots [1]

Photo 4.12 Wicker Works Three Months after Planting [1]

(2) Reinforcement of the Canal Bed Surface for Main Irrigation Canal (soil cement lining)

A soil cement lining 30 cm or more thickness is applied on the main irrigation canal bed in the PMS method irrigation project to reinforce and control seepage, ensuring that the water flow capacity of the main irrigation canal remains constant, considering that the lining is not easily removed by the flow of water. Although the cement composition depends on the nature of the ground, proper fixation can be achieved at 150 to 200 kg/ m^3 on the gravel ground, 200 kg/ m^3 with silty clay on the canal bed and 100 kg/ m^3 with silty clay in the canal corner. Since the canal bed portion requires more strength like a floor slab to some extent, the amount of cement composition is larger than that of the canal corner portion. When the ground in the canal foundation is soft, the soft soil is replaced with quality soil containing the proper amount of sand and clay and a soil cement lining is applied following sufficient compacting. The situations of soil cement lining are shown in Photo 4.13 and Photo 4.14.

Photo 4.13 Finishing Work by Soil Cement Lining [3)]

Photo 4.14 Canal Corner Filled by Soil Cement [3)]

The foundation surface of main irrigation canal may be dropped out by piping under the canal bed caused by water flowing in from external parts as well as water seeping from the canal bed. Such external water may be caused by water seeping into the canal side due to the rise in the water level in the main irrigation canal along the river, rainwater seeping into the canal bed from the slope surface in the canal side, if applicable, or leakage from the adjacent regulating pond. When such external water seepage is expected, the main irrigation canal bed foundation shall be carefully treated. In addition to the infiltration of water, as shown in Photo 4.15, there are cases where the irrigation canal bed collapses due to improper foundation conditions, so confirmation before construction work is required.

Photo 4.15 Collapse of Canal Bed due to Improper Foundation Condition [1)]

(3) Erosion of Gabion Revetment of the Main Irrigation Canal and Stability Against Back-earth Pressure

The gabion revetment of main irrigation canal shall not be eroded by the flow and shall remain stable against the back-earth pressure causing falling (rotation), sliding and ground bearing. The gabion revetment of the main irrigation canal in the PMS method irrigation project ensures structural stability as follows:

- Stability to withstand erosion of the revetment for main irrigation canal: the flow velocity within the steep gradient main irrigation canal is set as 1.7 to 1.8 m/s which is sufficiently below the allowable design velocity of the multiplied gabion revetment of the main irrigation canal (around 5 m/s).

- Stability to the back-earth pressure of the revetment for main irrigation canal: since the height of the gabion revetment of the main irrigation canal is less than 5 m, it remains fully stable against falling (rotation), sliding and bearing provided the back of the revetment is properly backfilled.

4.3.5 | Design of the Water Wheel in the Main Irrigation Canal

(1) Purpose of the Water Wheel

Any farmland located at an elevation higher than the canal, or even along it, cannot usually benefit from the canal. However, using a water wheel allows water to be conveyed into such farmland if the elevation difference is 3 to 5 m or less, which is also expected to encourage farmers to participate in maintaining and managing the canal. Meanwhile, disordered installation will reduce the canal flow discharge amount as well as decreasing the flow velocity, hindering the required amount of water from reaching downstream. Also, the irrigation canal themselves are very vulnerable to damage the canal bed by installation of water wheel. Accordingly, when installing a water wheel, care shall be taken to fully consider that the water wheel is not installed chaotically and without permission by discussing with the water users' association or irrigation association which oversees maintenance and management of the irrigation facilities.

(2) Design of the Water Wheel

The water wheel is installed in a place with an appropriate flow velocity/discharge amount for its operation, such as downstream of the drop works. The following shall be noted for the water wheel construction:

- A wheel made of iron and partly aluminium alloy is applied. For the shaft and bearing, which will be the most prone to wear, a shaft used as a small hydraulic power generator is diverted.
- A bucket is attached to both sides of the water wheel and an iron U-shaped ditch is combined to convey the water away.
- Since the flow around the water wheel is likely to fluctuate and riverbed erosion may occur, it shall be considered that the wheel body is likely to be damaged by erosion of the post foundation supporting the wheel bearings.

4.3.6 | Effects of Vegetation Works along the Main Irrigation Canal

Wicker works are applied to reinforce the gabion revetment on both banks of the main irrigation canal. Other than this, vegetation works are useful for many purposes, including as windbreaks, erosion control forest, flash flood and debris flow control and for protecting the external embankment walls. Wickers are suitable for protecting the gabion revetment of the river and canal; Tamarisk and eucalyptus for the erosion control forest and eucalyptus for protecting the embankment slope with much seepage water. In addition, olive, viyella and mulberry are usable for embankment slope protection.

Eucalyptus grows extremely fast while the tree hinders visibility. In cropland areas, it blocks sunlight, affecting crop raising. And Eucalyptus is effective as windbreaks to prevent sedimentation caused by wind in the canal and improve the environment of human activity by mitigating hot air and sandstorms from deserts. Table 4.8 and Photo 4.16 shows the list of vegetation works frequently applied in existing PMS irrigation projects.

Table 4.8 List of Vegetation Works Frequently Applied [3)]

Plant	Tree height	Characteristic	Planting method	Purpose
Wicker	5 to 7m	Thrives at waterside	Cutting	Wicker works, bank protection
Tamarisk	10 to 15m	Thrives in a dry desert	Cutting	Windbreak, erosion control forest
Eucalyptus	10 to 15m	Thrives in wetland	Raising seedlings and transplanting	Windbreak, erosion control forest, revetment
Olive	1.5m to 3m	Suitable for dry areas, deep-rooted	Transplanting young saplings	Embankment slope protection
Mulberry	5 to 10m	Withered by over-humidity	Transplanting young saplings	Embankment slope protection
Shisham	8 to 15m	Grows naturally at riverside	Transplant naturally grown young tree	Bank protection, firewood production
viyella	3 to 5m	Grows naturally in a desert	Raising seedling and transplanting	Embankment slope protection

Flash flood control by a Eucalyptus plantation zone

Tamarisk applied as a windbreak in a desert

Slope protection by Eucalyptus

Dike surface slope protection by *Shisham*

Photo 4.16 Vegetation Works Frequently Applied[3)]

4.4 Sand Basin (Regulating Pond) Design

4.4.1 | Basic Policy for Designing the Sand Basin (Regulating Pond)

The following priorities shall be taken into account when designing the sand basin (regulating pond). The procedures for the sand basin design are shown in the design process in Figure 4.37.

- The sand basin (regulating pond) must not be excessively deep and preferably around 2 m, to secure workability and conduct carrying out of accumulated sediment.
- A regulating pond is provided to facilitate interchange among the canal branch/junction point and gradient change points of multiple canals.

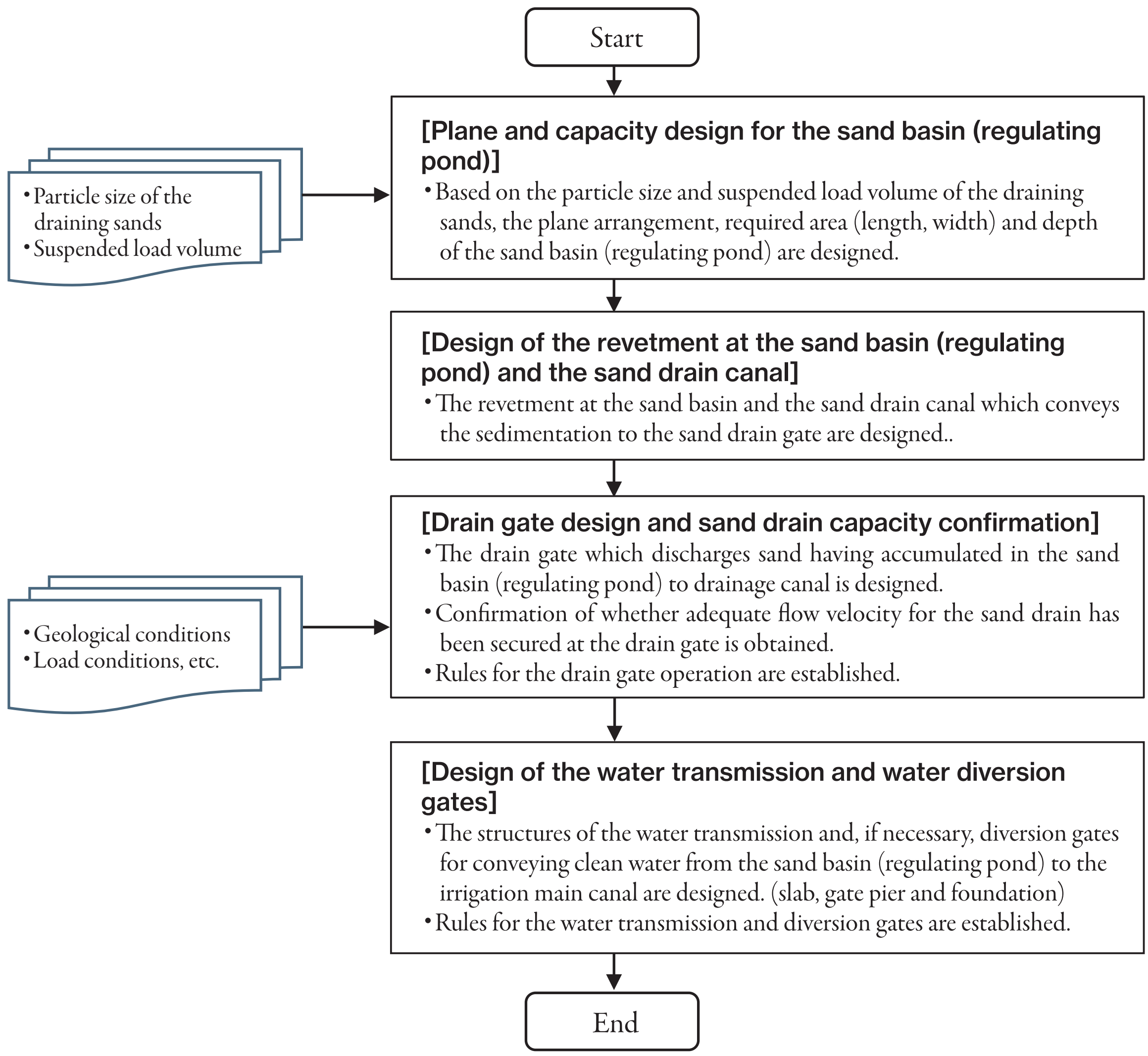


Figure 4.37 Design Process for the Sand Basin (Regulating Pond) [2)]

4.4.2 | Type and Design Polity for Sand Basin (Regulating Pond)

(1) Sand Basin (Regulating Pond) Type

A sand basin is provided at the end of the steep gradient main irrigation canal, comprising the revetment surrounding the basin, the sand drain canal at the bottom, drain gate and water transmission and diversion gates. A regulating pond is provided where the main irrigation canal and existing water channel come together, and its function and structure are the same as the sand basin. While the sand basin (regulating pond) can be formed by placing concrete on all sides, the PMS method irrigation project adopts the stone masonry and gabion revetment method, and a concrete sand drain canal installed at the bottom. Moreover, a manual slide gate with flush board is equipped for the water transmission and diversion gates.

The stone masonry and gabion revetment method is affordable compared to a sand basin placing cast-in-place concrete on all sides. Since it can be established by materials (stones, annealing wires for gabion, bricks, cement, woods, iron plate, etc.) that are available in Afghanistan, local residents are fully capable of maintaining and managing the facility. Table 4.9 shows a comparison of the types of sand basin.

Table 4.9 Sand Basin (Regulating Pond) Types

The stone masonry and gabion revetment method with a concrete sand drain canal on the basin bottom (applied as a PMS method irrigation facility)	Placing concrete on all sides (adopted in many countries)
 Source of photo: 1)	 Source of photo: JICA

(2) Design Policy of the Sand Basin (Regulating Pond)

The design specifications of the sand basin (regulating pond) in the existing PMS irrigation project are shown as follows. Given the need to maintain and manage the sand basin, the following depth and installation interval of the sand basin (regulating pond) shall constitute the standard design of the PMS method irrigation project. Meanwhile, the sedimentation particle size, area/capacity of the sand basin and specifications of the drain gate/water transmission and diversion gates are designed in accordance with regional conditions, such as the irrigation discharge and suspended load volume in the target region, while benchmarking the following actual specifications from past projects. Design drawings of the sand basin are exemplified in Figure 4.38 to Figure 4.43.

- Particle size of sedimentation:	Sand 0.08 mm or larger
- Sand basin depth:	Around 2 m (with maintenance and management in mind)
- Installation location of the sand basin:	Within approximately 1 km from intake mouth
- Area and capacity of the sand basin:	1,100 to 1,600 m^3
- Drain gate and water transmission/diversion gates:	Drain gate: 1.6 x 1.6 x 1 gate Water transmission gate: 1.5 m width x 3 gates Water diversion gate: 1.5 m width x 1 gate

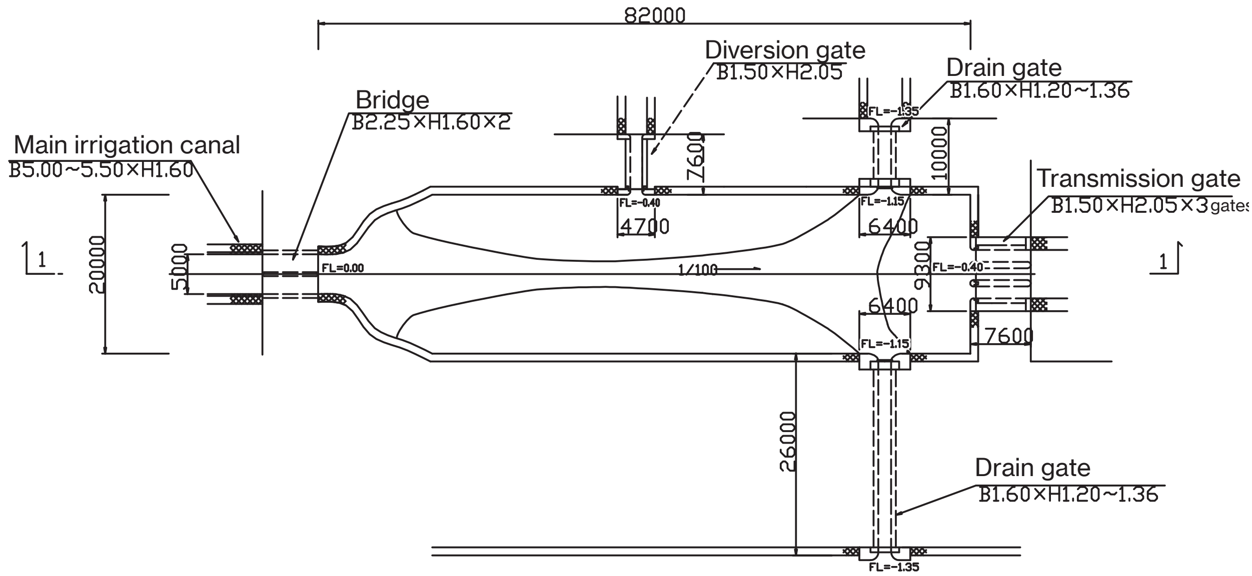


Figure 4.38 Example Plane Drawing of the Sand Basin [3)]

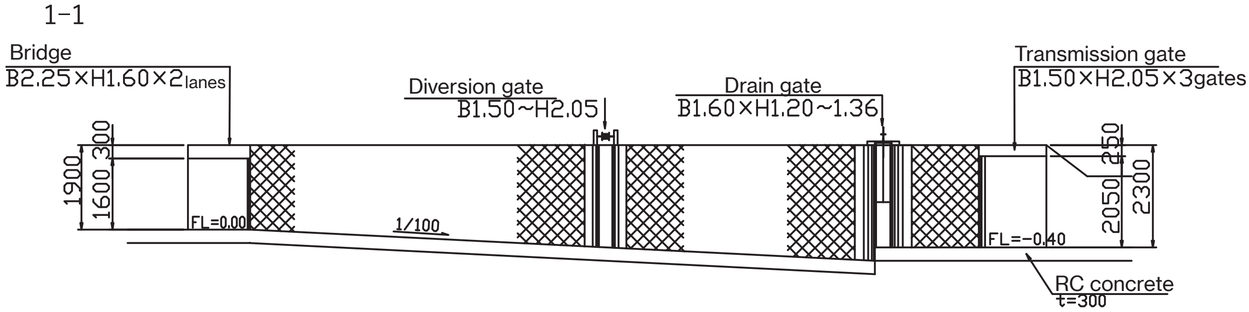


Figure 4.39 Example Profile Drawing of the Sand Basin [3)]

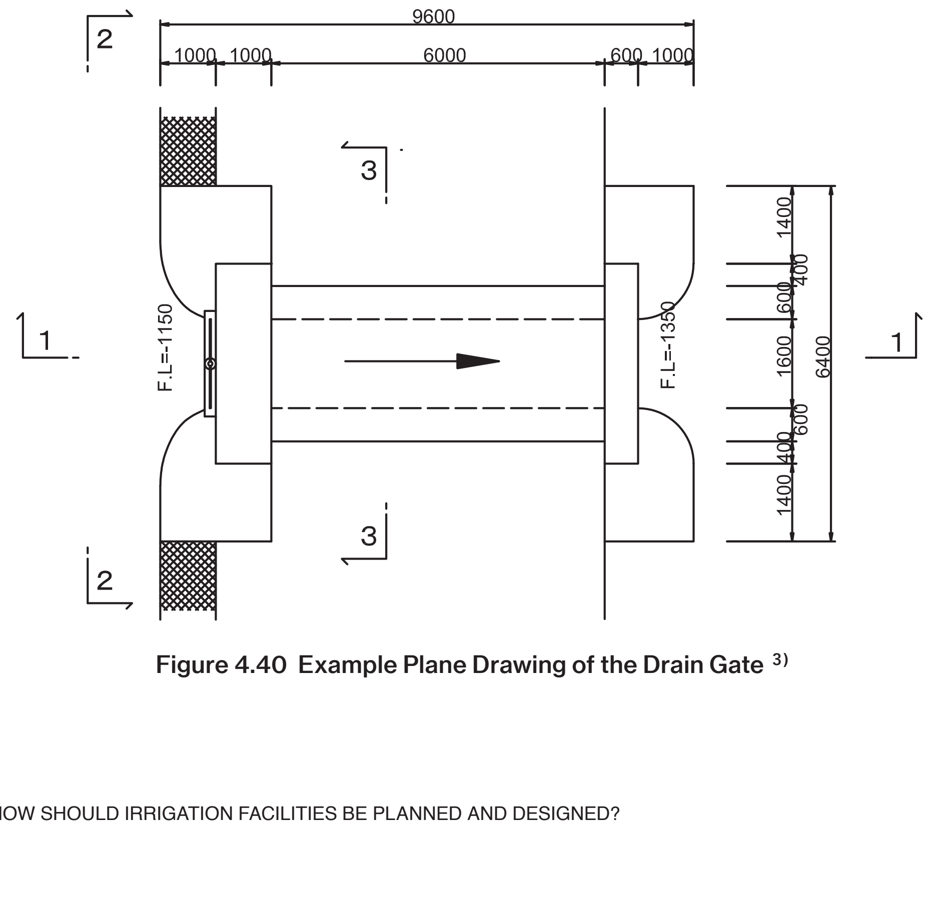


Figure 4.40 Example Plane Drawing of the Drain Gate [3)]

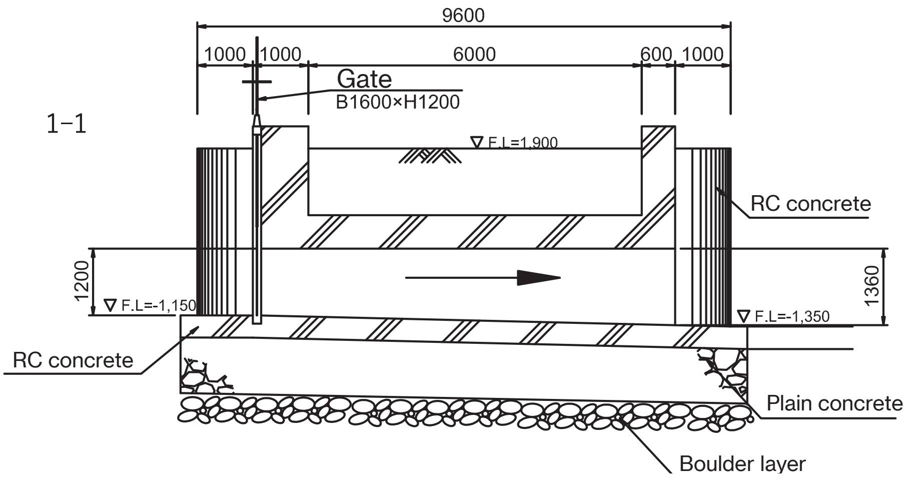


Figure 4.41 Example Profile Drawing of the Drain Gate [3)]

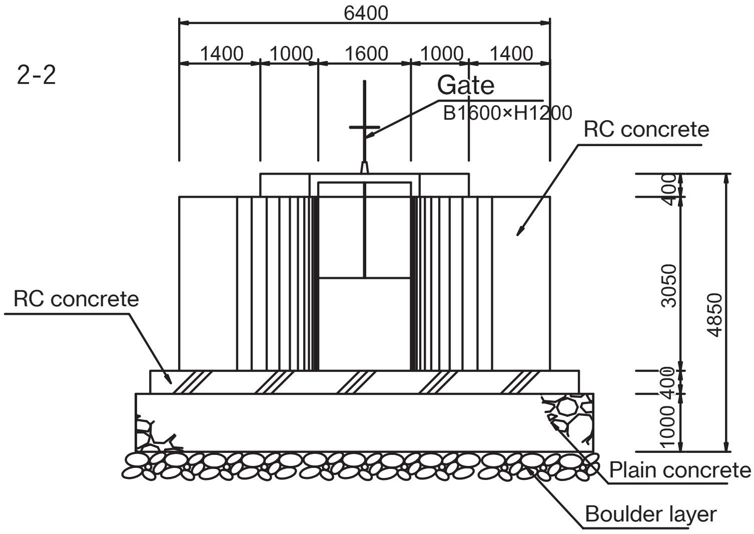


Figure 4.42 Example Front View Drawing of the Drain Gate [3)]

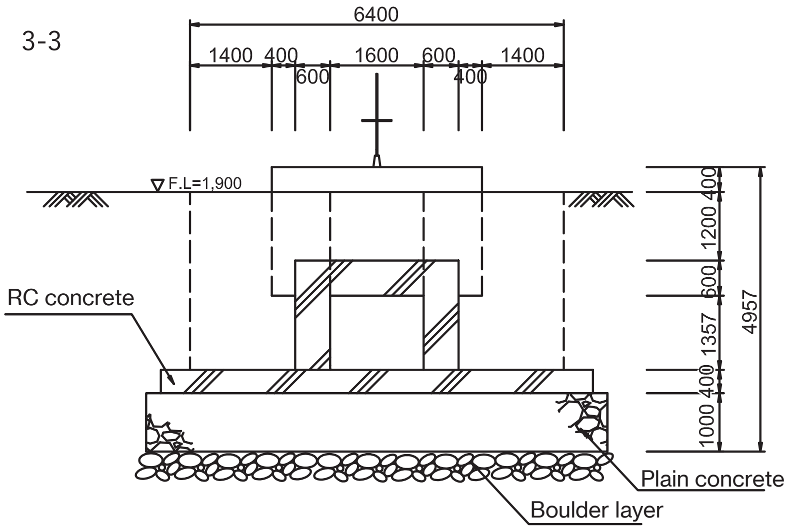


Figure 4.43 Example Cross-sectional Drawing of the Drain Gate [3)]

4.4.3 | Specification Design of the Sand Basin (Regulating Pond)

The sand basin has the role to settle any inflowing sand. And the surface loading factor is an indicator representing this settling function. When the inflow rate to sand basin is *Q* and the sand basin area is *A*, the surface loading factor (V_0) is calculated as $V_0 = Q/A$, meaning "the sinking speed at which sand particles flow in from the upper end of the sand basin is accumulating at the bottom of the basin outlet". Here, when the sinking speed of the actual particles is *U*, the sand removal rate is U/V_0. Therefore, to enhance removal performance, possible methods include enlarging the sinking area (*A*) in the sand basin, reducing the flow rate (*Q*) and increasing the sand sinking speed (*U*) (target particle size).

As above, the sand basin is designed by the following method based on the target particle size of the sand settled, flow velocity in the sand basin and surface loading factor (see Figure 4.44).

- The sand basin is provided at the end of a steep gradient main irrigation canal. However, amid considerable inflow sediment, there may be a need to provide multiple sand basins given the required maintenance frequency. Even where the capacity of the sand basin cannot increase sufficiently due to land restrictions, installation of multiple sand basins is planned.
- The sand particle size to be settled shall be 0.08 to 2 mm and silt and clay with smaller particles will not settle. Since they sink very slowly, most are suspended and do not subside to the riverbed. Usually, settling sands remain in the water for 10 to 20 minutes. Planning a sand basin to encourage this silt and clay to subside would involve an excessive facility.
- A cross-sectional shape (width and depth) of the sand basin is determined to achieve an average flow velocity of 2 to 7cm/s in the sand basin. It is empirically proven that the settled sands do not resurface when the average velocity is maintained at this level. Moreover, the sand basin shall be around 2 m depth given its maintenance and management. When enlarging the sand basin area, the depth becomes shallow and vice versa. Although the area shall be reduced in the case of land restriction, maintenance and management - including sand dredging - will be easier when enlarging the area and reducing depth.
- The length of the sand basin is calculated by the following formula and the relation between the sand particle size and sinking speed is shown in Table 4.10. Since the scope includes sand particle sizes up to 0.08 mm, when the sinking speed (*U*) is 0.6 cm/s (0.006 m/s), valid water depth (*H*) 2.0 m and average flow velocity in the sand basin (*V*) is 7 cm/s (0.07 m/s)m the required sand basin length is calculated as $1.5\ to\ 2.0 \times \left(\frac{2.0}{0.6} \times 7\right) = 35\ to\ 47m$. Thus, the sand basin length is determined as 40 to 50m.

$$L = K \times (H/U \times V) \quad \cdots\cdots (4.17)$$ 2), see 12)

Here, *L*: sand basin length (m), *K*: coefficient (safety ratio) *K*=1.5 to 2.0, *H*: valid water depth (m), *U*: sinking speed of sand to be removed (cm/s), *V*: average flow velocity in the basin (cm/s)

Table 4.10 Sand Sinking Speed 2), see 12),13)

Particle size	Sinking speed	Sinking speed
(mm)	(cm/s)	(m/s)
0.30	3.2	0.032
0.20	2.1	0.021
0.15	1.5	0.015
0.10	0.8	0.008
0.08	0.6	0.006
0.06	0.3	0.003
0.04	0.1	0.001
0.02	0.03	0.0003
0.01	0.01	0.0001

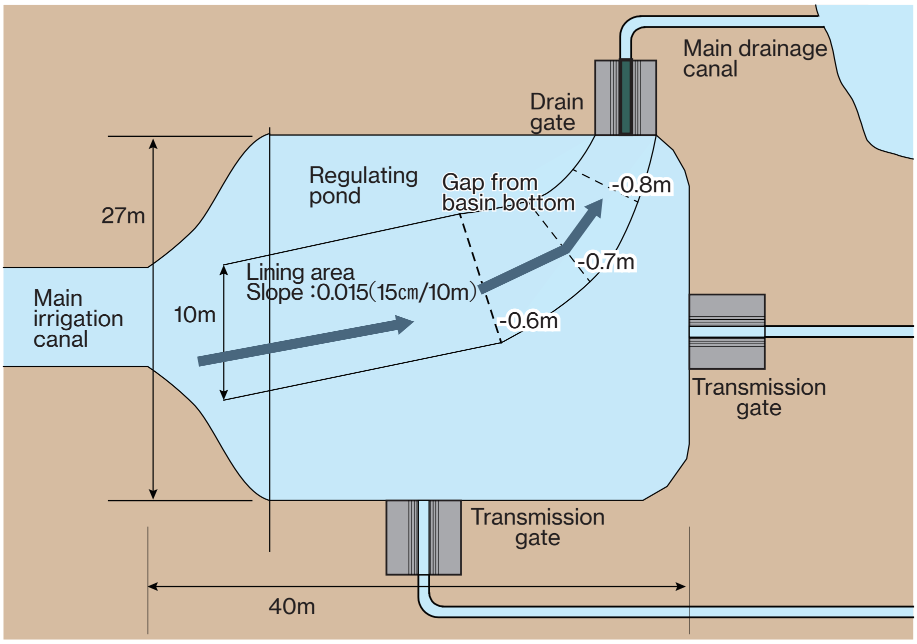
- Assuming a 100% sand removal ratio in the sand basin, the sinking speed is 0.6cm/s (0.006 m/s) when the sedimentation particle size is 0.08 mm or larger. Accordingly, the required sand basin area is calculated as the surface loading factor :   is set to 0.6 cm/s (0.006 m/s). When the particle size exceeds 0.08 mm, the surface loading factor (V_0) can be increased while reducing the sand basin area (A). Conversely, when a smaller particle size is needed, the surface loading factor (V_0) shall be decreased while enlarging the sand basin area (A).
- Dredging in the sand basin shall be carried out each time the sedimentation depth becomes around 50 cm to prevent the intake suspension period from being too long due to dredging works. The daily sedimentation volume in the sand basin is determined using the sand concentration obtained by the sediment discharge survey (Chapter 3) as follows:

$$V = \frac{d \times (Q_i - Q_d) \times 60 \times 60 \times 24}{1900} \quad \cdots\cdots \quad (4.18)^{2)}$$

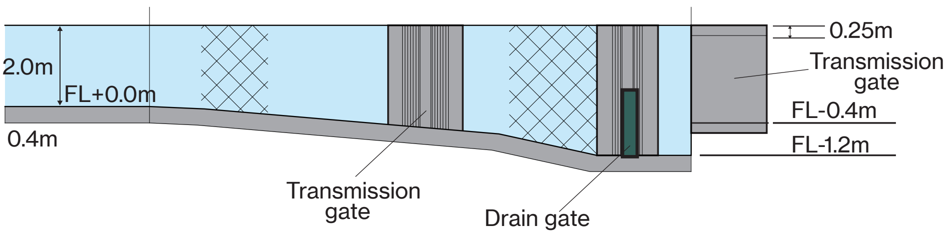
$$F = \frac{0.5 \times A}{V} \quad \cdots\cdots \quad (4.19)^{2)}$$

Here, V: daily sedimentation volume (m^3/day), d: sand concentration (g/ℓ), Q_i: intake discharge amount (m^3/s), Q_d: drain discharge amount (m^3/s), F: dredging frequency (day), A: sand basin area (m^2), unit sedimentation weight: 1900g/m^3, sedimentation depth: 0.5m

As shown in the above formulas, the higher the sand concentration and intake discharge amount, the more the sedimentation volume increases. Accordingly, the dredging frequency becomes more. In addition, the larger the sand basin area is, the less the dredging frequency.



Plane schematic drawing of the sand basin (regulating pond)



Profile schematic drawing of the sand basin (regulating pond)

Figure 4.44 Plane and Profile Schematic Drawings of the Sand Basin [3]

Text Block 4-3: Dredging Frequency of the Sand Basin

The sand basin in the Miran project applied the following specifications described on "Green Ground Project". According to the record of existing PMS irrigation project, the sedimentation volume in the sand basin in summer in Miran was 450 m^3 per week, requiring weekly dredging works, while dredging works in winter were required every two months.

Table Sand Basin Specifications in the Miran Project [3)]

Weir	Specifications	Capacity	Intake discharge amount
Miran	27m × 40m × 2m	2,160 m^3	Drought period: : 2.0 m^3/s Flood period: : 4.0 m^3/s

As above, dredging volume on flood period is around 20% of the sand basin capacity (=450/2,160) and the sand depth around 40 cm (=450/(27x40)).
In this case, the sand concentration is presumed as follows:
When the daily sedimentation volume as = 450 ÷ 7= 64.3 m^3/day and the unit sand weight as 1,900g/ m^3, drainage volume from drain gate as 1.53 m^3/s (opening height of drain gate is 30cm), the sand weight is
64.3 m^3/day x 1,900g/ m^3 = 122,143g
Here, the sand concentration is presumed as:
122,143g x 1,000g / ((4 m^3/s − 1.53 m^3/s) x 60s x 60min x 24hr x 1000 ℓ)= 0.57 g/ ℓ

4.4.4 | Structural Design of the Sand Basin (Regulating Pond)

(1) Design of the Surrounding Revetment

Revetment is constructed by stone masonry and gabion works in the sand basin (regulating pond). The design method of revetment shall be based on "4.3.4 Structural Design of the Main Irrigation Canal".

(2) Design of the Concrete Sand Drain Canal

At the bottom of the sand basin (regulating pond), a concrete sand drain canal is provided with a gradient of 1/70 to 1/100 and around 10 m width to flush out the sand-mixed water in the sand basin by conveying the water to the drain gate at a flow velocity exceeding 3m/s (see Photo 4.17).

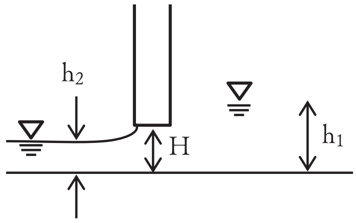
Photo 4.17 Concrete Sand Drain Canal at the Bottom of the Sand Basin and Drain Gate [3)]

(3) Design of the Drain Gate

The drain gate of the sand basin (regulating pond) is the sliding gate, discharging the bottom water and sand from the sand basin to reduce dredging frequency. The drain gate base elevation is 80 cm lower than the basin bottom to convey sand to the drain gate. As shown in Figure 4.41, the sand-mixed water discharged from the drain gate is conveyed to the river or main drainage canal via the reinforced concrete box culvert. This box culvert is designed considering earth pressures on the upper and side parts and the vertical load of vehicles, etc. A direct foundation with a depth of about 1 m will be constructed under the culvert, and a reinforced concrete bottom slab with a thickness of 40 cm or more will be constructed above the foundation.

Photo 4.18 and Figure 4.45 show a front view of the drain gate in the sand basin (regulating pond) and a sand drain image. The opening in the lower part of the drain gate discharges water like an orifice, thereby flushing sands accumulate at the bottom of the basin. The drain discharge amount from the bottom is calculated by the following formula used to calculate outflow from a gutter gate/culvert.

The above-mentioned flow velocity in the sand basin described in 4.4.3 is secured by draining a certain amount of the water flowing into the sand basin from the drain gate. The opening height of the drain gate shall ensure that the required amount of irrigation water downstream from the sand basin is secured, and that the minimum flow velocity about 2 cm/s in the sand basin shall be secured when the water in the sand basin is drained from the drain gate. The calculation result is also applied to consider the dredging frequency of the sand basin (regulating pond) as shown in 4.4.3.



Submerged outflow: $h_2 \geqq H$ $Q = CBH\sqrt{2g(h_1 - h_2)}$ $C = 0.75$ .... (4.20) [2), see 14)]

Subsurface outflow: $h_2 < H$ and $h_1 \geqq 3/2H$ $Q = CBH\sqrt{2gh_1}$ $C = 0.51$ .... (4.21) [2), see 14)]

Free outflow: $h_2 < H$ and $h_1 < 3/2H$ $Q = CBh_2\sqrt{2g(h_1 - h_2)}$ $C = 0.79$ .... (4.22) [2), see 14)]

However, when free outflow is $h_1 / h_2 \geqq 3/2$, it is replaced to $h_2 = 2/3\, h_1$.
Here, H: height of sluice gate/culvert, B: width, h_1: deeper water depth among those measured from the outflow base elevation, h_2: shallower water depth

Photo 4.18 Drain Gate Outlet of the Sand Basin and Drain Gate Outflow from the Sand Basin [1)]

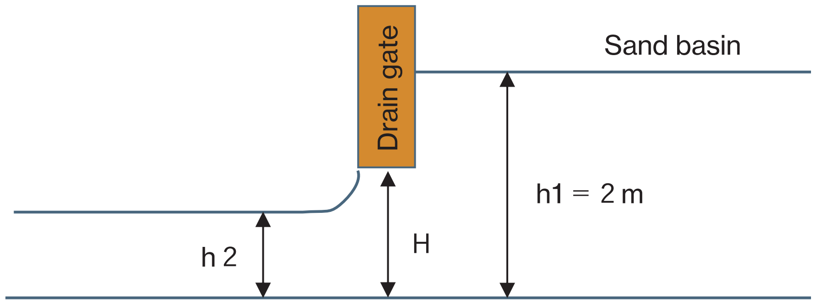


Figure 4.45 Image of Water Discharge from the Drain Gate [2)]

(4) Design of the Water Transmission/Diversion Gates

The water transmission gate and diversion gate control the outflow of water in the sand basin by flush boards and convey the top water to the main irrigation canal. The sand-mixed water drawn at the intake gate is conveyed to the sand basin via a steep gradient main irrigation canal. Accumulating the sand means the top surface water becomes clean and the design method for the water transmission/diversion gates are the same as "4.2.6 Design of the Intake Gate".

4.5 Design of the Reservoir, Siphon, and other Facilities

4.5.1 | Basic Policy for the Design of the Reservoir, Siphon, and other Facilities

When designing the reservoir, siphon and other facilities, the priorities are shown as follows. The procedures for designing the reservoir, siphon and other facilities are shown as the design process in Figure 4.46.

- In the case of the following topographical/ geographical conditions, a siphon, rather than a reservoir, shall be provided as a general rule, since the water flow entails a high risk of collapse.
  - Where the wadi or river basin area is large; or
  - Where there is a permeable thick sand layer in the foundation on which the dike is arranged, and the risk of the dike breaking emerges due to a water channel formed in the foundation; or
  - In case there is a sign of landslide on the slope of the abutment for reservoir; or
  - Although a wider embankment is required if the water level is abnormally high, the required embankment width cannot be secured due to land restrictions.
- The reservoir is installed when the wadi or river basin is small, the total flow discharge amount during a flash flood or debris flow is relatively smaller and the reservoir capacity is sufficient to absorb the flow. In other words, the reservoir serves as an erosion control dike. When considering the adoption of a reservoir, the runoff amount will be calculated in consideration of the basin area on the slope on the mountain side of the reservoir. And it is necessary to confirm whether or not the capacity of flash flood and debris flow can be absorbed by the freeboard of the reservoir and the freeboard of the main irrigation canal at downstream section of the reservoir.
- The siphon is provided in the main irrigation canal after the stage flowing through the sand basin to ensure it is not buried or blocked by sand in the raw intake water. Further, a protective net shall be provided on the top of vertical shaft of the siphon to prevent from falling inside the shaft.
- When the flood water is small in scale and the flow overpasses the main irrigation canal with certain limited flow width, a flood crossing bridge is installed.

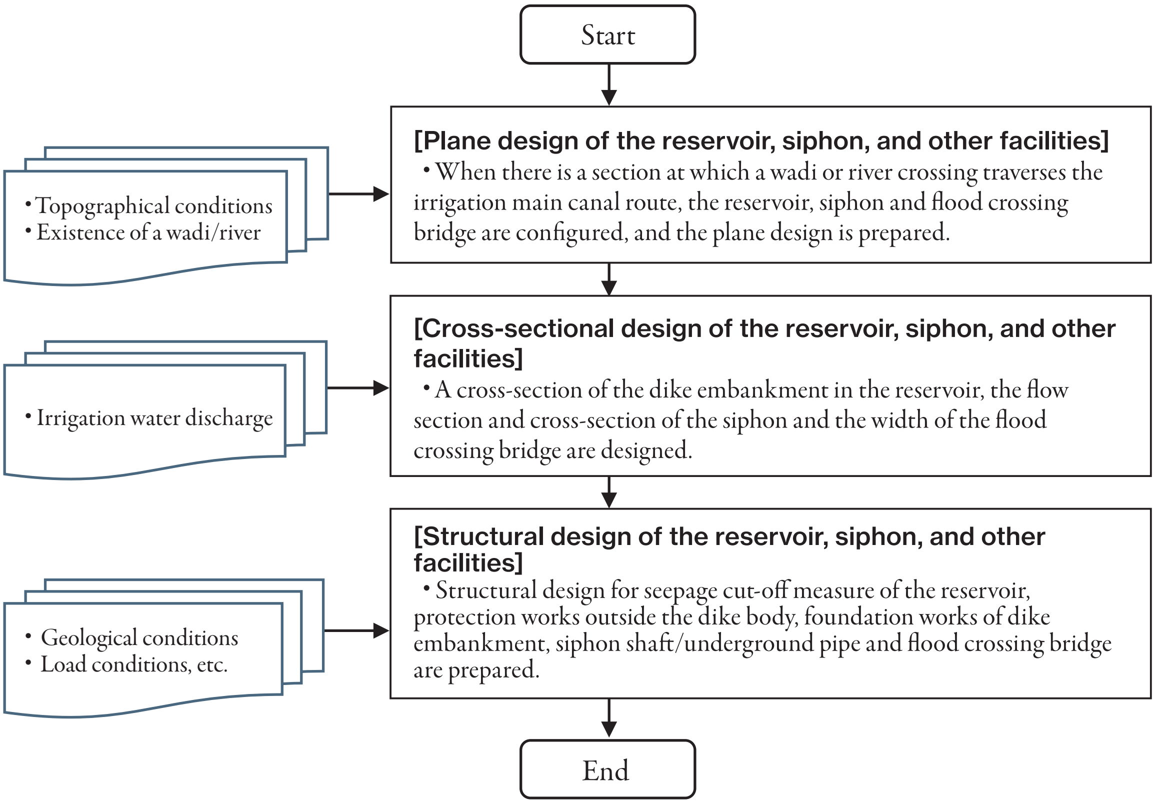


Figure 4.46 Process for Designing the Reservoir, Siphon and other Facilities [2)]

4.5.2 | Type and Design Policy for the Reservoir, Siphon, and other Facilities

(1) Type for the Reservoir, Siphon, and other Facilities

The reservoir comprises a dike embankment, blanket works, works to protect the external slope of the dike, drain works and crown protection works, etc. Although the reservoir may be a concrete structure, the earth dike is the standard in the PMS method irrigation project. The embankment of the reservoir is constructed by proper quality sandy soil with local surplus soil. The water cut-off applying cohesive soils is provided on internal dike slope surface of the reservoir. The external dike slope is protected by vegetation and drainage treatment of rainwater and seepage water is applied by laying permeable gravel.

The earth dike is affordable and needs no solid foundational ground compared with a concrete structure. Since the relevant materials are available in Afghanistan, local residents are fully capable of maintaining and managing the facility. Since the siphon and flood crossing bridge are exposed to earth and water pressures and vertical loads, they need a rigid structure, usually concrete. Accordingly, a concrete structure shall also be adopted in the PMS method irrigation project. Table 4.11 shows a comparison of the types of reservoir.

Table 4.11 Reservoir Types

Earth structure (applied as a PMS method irrigation facility)	Concrete structure (adopted in many countries)
 Source of photo: 3)	 Source of photo: JICA

(2) Design Policy of the Reservoir, Siphon, and other Facilities.

The design specifications of the reservoir, siphon and other facilities in the existing PMS irrigation project are shown as follows. To establish a water cut-off, water drain, and slope and crown protection of the reservoir embankment structure, the following reservoir structure shall constitute the standard design for the PMS method irrigation project. Meanwhile, other reservoir canal specifications are designed in accordance with regional conditions, such as the topographical conditions and features including wadi and river and design irrigation discharge while benchmarking the following actual specifications from past projects. Figure 4.47 shows an example cross-sectional drawing of the reservoir and Figure 4.48 shows an example plain and cross-sectional drawings of the siphon.

- Reservoir structure :	An earth dike constructed by sandy soil mixed with boulders, blanket works using cohesive soil, slope protection in the external dike by vegetation and drainage treatment by laying permeable gravel, crown protection
- Reservoir length and width :	150 to 350 m length, 100 to 250 m width
- Reservoir depth :	5 to 8 m
- Reservoir width and height :	100 to 150 m width, 15 m length
- Siphon length and cross-section :	20 to 200 m length, cross-section: 1.2 x 1.2m
- Flood crossing bridge length and width :	30 m or more length, bridge width equivalent to irrigation canal width

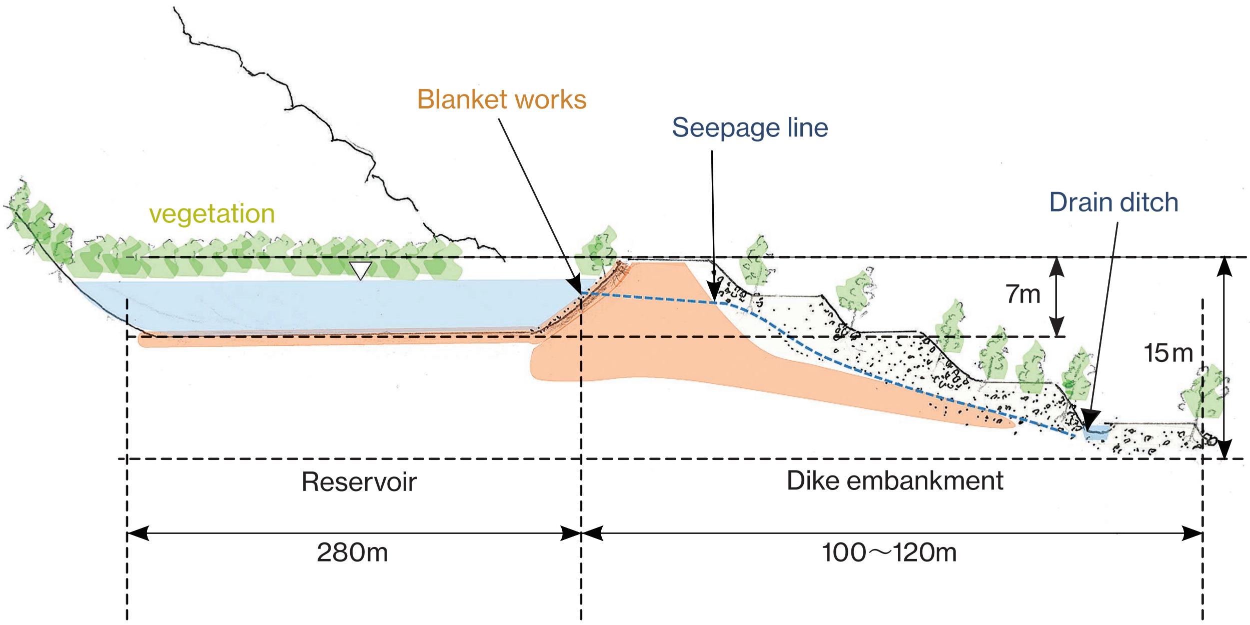


Figure 4.47 Example Cross-sectional Drawing of the Reservoir [3)]

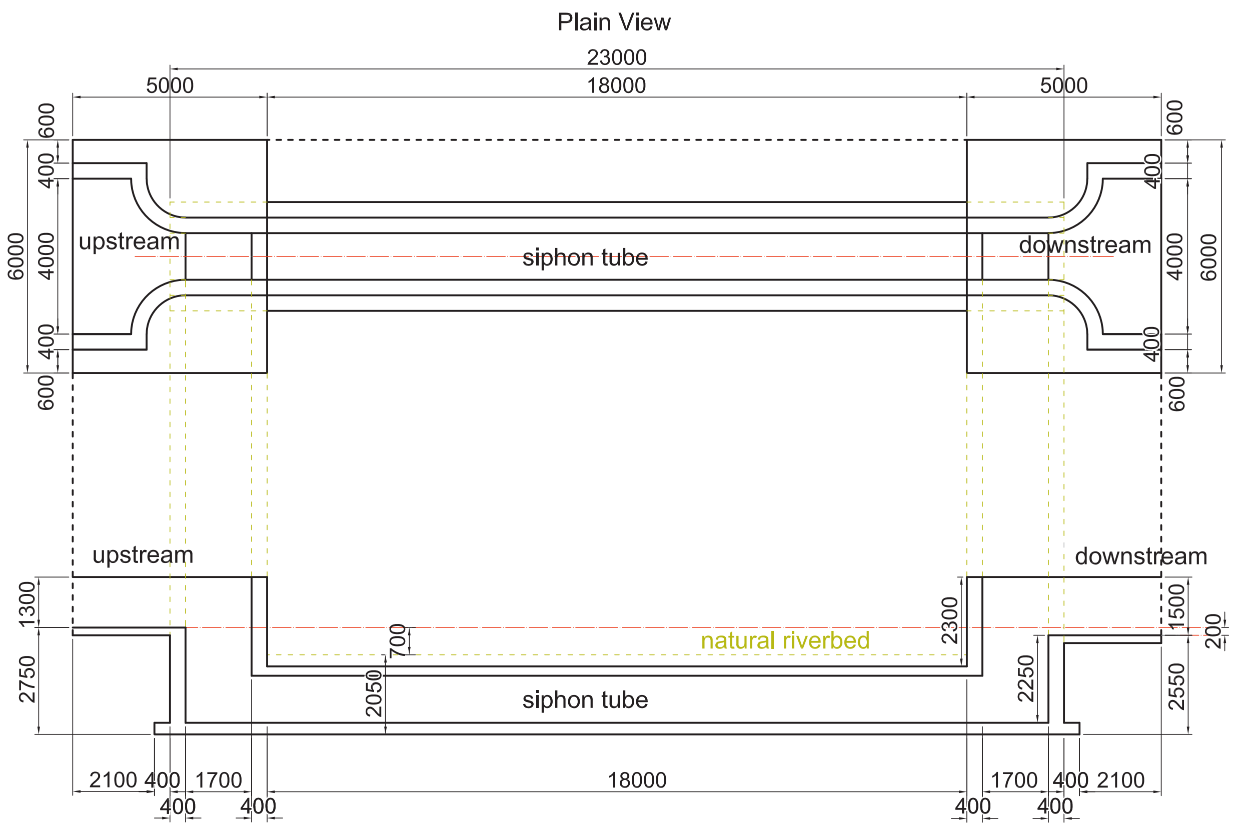


Figure 4.48 Example Plain and Cross-sectional Drawings of the Siphon [1)]

4.5.3 | Plain Design of the Reservoir, Siphon, and other Facilities

In many cases, the main irrigation canal extending over several kilometer will pass along the foot of the mountain, cross the valley, and traverse wadi and rivers prone to flash flood and debris flow. When the main irrigation canal passes along the foot of a mountain where rocks are exposed, the main irrigation canal is laid on a large-scale embankment constructed on the valley side to avoid excessive labor and costs for bedrock excavation. When traversing a small but steep gradient valley, a reservoir is established to take the flow from the valley as a means of countering flash flood/debris flow. This reservoir also serves as irrigation during periods of drought and adjusts the water distribution. When the valley is small and flash flood/debris flow is expected to be on a small scale, a flood crossing bridge can also be provided. When the valley is large and the gradient is gentle, part of the main irrigation canal section will be provided as a siphon to protect the canal against flash flood and debris flow. The plain design method for the reservoir, siphon, and other facilities is described as follows:

- A reservoir is installed in a section at the foot of the mountain where a steep slope on the mountain side is adjacent to the route of the main irrigation canal and where flash flood and debris flow flows down a steep slope at a gradient of 15° or more as shown in Figure 4.49. Nevertheless, where the valley terrain with the slope on the side of the main irrigation canal has gentle gradients, a siphon or flood crossing bridge is installed as shown in Figure 4.50. The photo of a siphon and a flood crossing bridge are shown in Photo 4.19.

Table 4.12 Countermeasure for Flash Flood and Debris Flow from Slope [2)]

Facility Type	Selection conditions
Reservoir	- In case a steep slope with a relatively small basin area is located close to the main irrigation canal route, and a small flash flood or debris flow is expected. - In case the site situation corresponds the conditions shown in "4.5.1 Basic Policy for the Design of the Reservoir, Siphon, and other Facilities", adoption of the reservoir shall be avoided.
Siphon	- In case the main irrigation canal crosses a river or wadi with a relatively gentle slope of a large basin area. And in case the water level of the flash flood is conspicuously high and a dike embankment is required along the main irrigation canal.
Flood Crossing Bridge	- In case the scale of the flash flood and debris flow is small and the width of them is limited.

- The slope line of the dike embankment in the reservoir is set as it connects interchanging points between the revetment crest of the canal and the mountain ridge, and those surrounded by the earth dike and valley constitute the reservoir area. The area and capacity of the reservoir shall be maximized.
- The section where the siphon or flood crossing bridge is installed is determined by checking the target area and situation of the valleys and slopes where flash flood and debris flow down via satellite images or getting feedback from local residents, and further by anticipating the scale of flash flood and debris flow in future to ensure they will not affect the main irrigation canal. If the flash flood and debris flow directions are distributed over a wide-ranging area, a training dike wall comprising an embankment and gabion on the mountain side is installed to gather the flash flood and debris flow. After the examination of the training dike, the section to install a siphon or flood crossing bridge is determined.

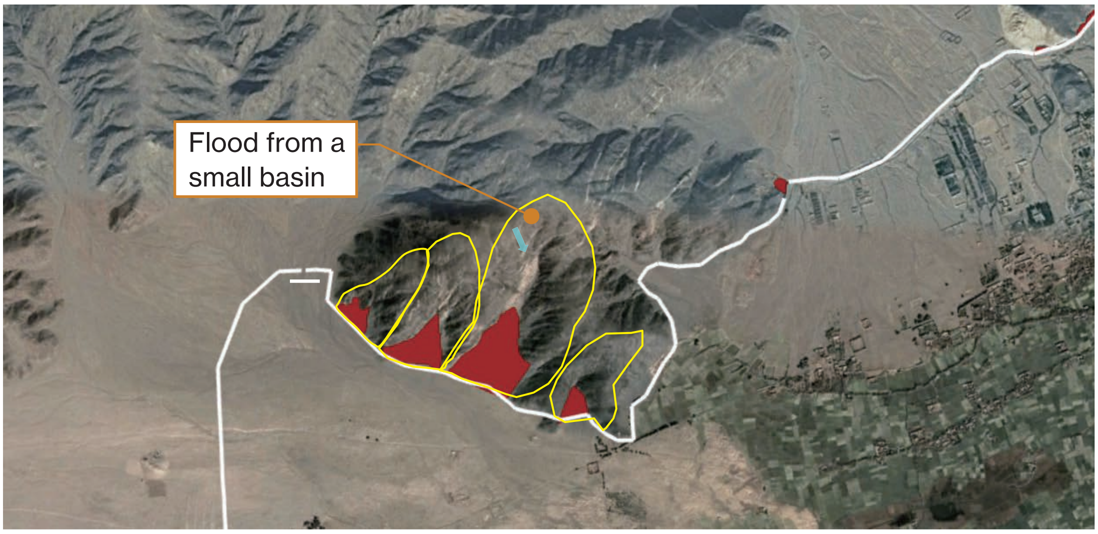


Figure 4.49 Reservoir Installation at the Foot of the Mountain (Marwarid Weir I) [2)]

Figure 4.50 Siphon Installation in the Flood Area [2)]

Photo 4.19 The Siphon (under construction) and Flood Crossing Bridge Crossing the Flood Area [1)]

4.5.4 | Cross-sectional and Structural Design of the Reservoir

The cross-sectional and structural design of the reservoir shall be as follows:

- The dike is constructed basically applying sandy soils which are available on site.
- The dike shall be as wide as possible in order to decrease the seepage line from the reservoir side.
- Where the embankment foundation is soft, the base is replaced with sands and gravel around 1.5 m thickness (sand mat method).
- A blanket (impermeable silt clay) is covered on the dike slope on the reservoir side.
- A drain is installed on the dike slope opposite the reservoir side by covering it with gravel and boulders to decrease the seepage line.
- The dike crown is covered in thick gravel to prevent the rainwater from softening the dike body.
- Trees are planted so as to surround the mountain side to which the reservoir is attached and the top of the reservoir, and the flow velocity of floods flowing into the reservoir is suppressed.
- To reduce the flow velocity and prevent destructive damage caused by unexpected overflow and flood, the retarding basin and broad vegetation zone are provided in the area where the foot of the embankment is in touch with the natural ground surface.

Figure 4.51 shows an illustration of the above.

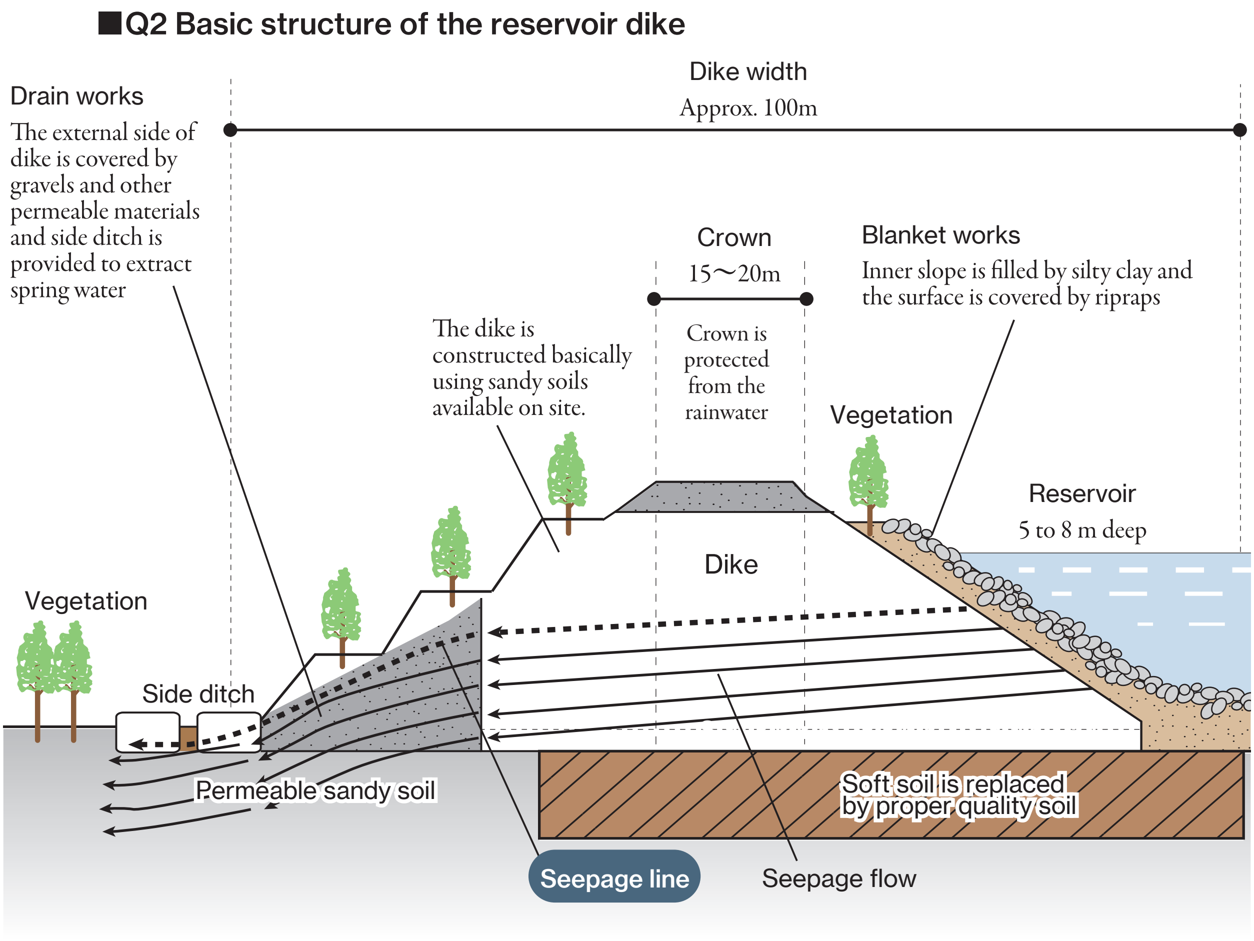


Figure 4.51 Basic Structure of the Reservoir by Earth Material [15)]

Text Block 4-4: Seepage Water from Reservoir and its Treatment (Report from Dr. Nakamura)

Leakage occurs in reservoirs because the bottom of the reservoir is higher than ground level and the water depth is deeper than in canals.

- The water in the reservoir permeates and flows through the dike as seepage water.
- When the water level is high, the water presents on the ground surface as "leakage" by infiltrating into the dike.
- When the ground is sandy, the water passes under the dike and flows out (boiling).
- In the case of a dike embankment, these water paths create a hollow through which water flows like a pipe (piping) .
- Accordingly, there is a need to construct a dike in a dessert valley covered by thick layers of sand.

From the above, the following construction methods of the reservoir shall be considered:

1) Cover the area inside the reservoir dike with materials impervious to water, such as silt clay (blanket works).
2) Lower the seepage line by making the dike body as thick as possible.
3) The external dike wall is reinforced by stone walls and tree planting and seepage water discharging ports are installed in the lower part of the gravel layer (drain works).
4) The top of the dike (crown) is covered in thick gravel to prevent rainwater from weakening the dike body.
5) In the case the foundation ground is covered with a thick layer of sand, it shall be replaced with proper quality soil containing an appropriate amount of clay, based on local surplus soil.

The above measures generally lead the satisfactory results. Since the seepage water volume is actually difficult to estimate, it is important to consider how to treat seepage water while monitoring the situation after the construction works.

4.5.5 | Cross-sectional and Structural Design of the Siphon

The cross-sectional and structural design of the siphon shall be as follows:

- A siphon comprises a shaft and underground pipe. The vertical shafts are provided on both sides of transverse obstacles and connected with the horizontal underground pipe or little inclined underground pipe toward downstream.
- The siphon structure prevents sedimentation by water in which the flow velocity in the siphon is 20 to 30% higher than the main irrigation canal directly upstream by reducing the cross-sectional area of underground pipes.
- A gate or stop log is installed on both upstream and downstream sides of the siphon shaft inlet as well as installing a falling prevention fence.
- An underground pipe is firmly inserted in the siphon shaft ensuring that no shafts slip out. Since the shaft and underground pipe are discontinuous structures, the pipe connecting area shall be backfilled and compacted to prevent different behaviours between the shaft and underground pipe and avoid any cracks in the connection part.
- A sedimentation pool 50 cm depth is provided on both upstream and downstream sides of the siphon shaft bottom given the purpose of preventing sand from flowing into the pipe and the workability of sediment removal from the shaft.
- The elevation gap in design water level between the upstream and downstream of siphon at the design

irrigation discharge shall secure the head loss (by hydraulic gradient, velocity head loss, freeboard) calculated by the following formula. The relation between the siphon structure and the head loss is shown in Figure 4.52.

$$H = i \cdot L + \beta \cdot \frac{V^2}{2_g} + \alpha \quad \cdots\cdots (4.23)$$ 2), see 16)

Here, H: siphon head loss (m), i: hydraulic gradient to the flow velocity in the siphon underground pipe, L: siphon underground pipe length (m), g: gravitational acceleration (= 9.81m/s^2), α : 50 to 80mm, β : 1.5 as the standard.

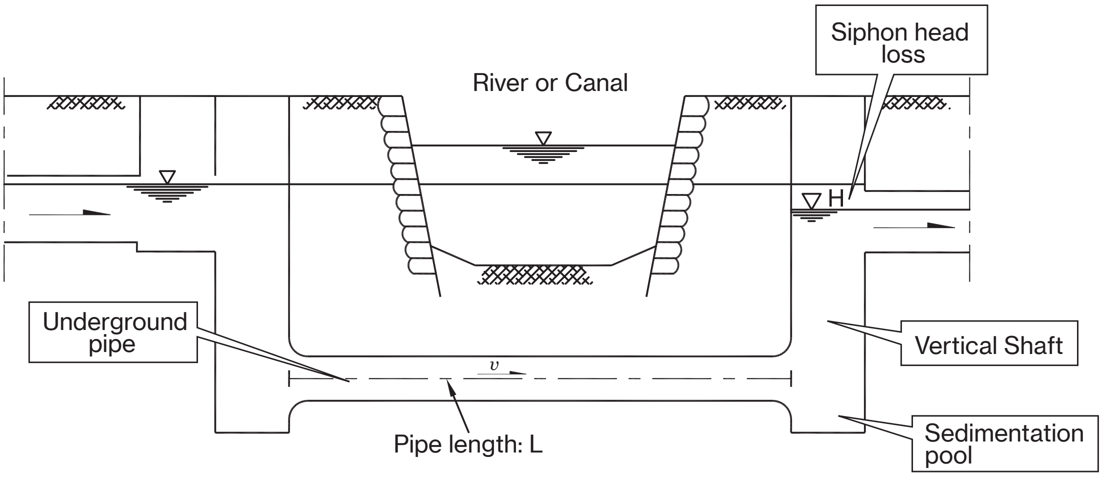


Figure 4.52 Structure and Water Level of the Siphon 2), see 16)

4.5.6 | Cross-sectional and Structural Design of the Flood Crossing Bridge

Like the siphon, the flood crossing bridge is provided in a place where the flood or debris flow transverses the main irrigation canal. The design of the flood crossing bridge shall be as follows:

- The flood crossing bridge is provided when a relatively small-scale flood overpasses the main irrigation canal and the flow width is limited. See Figure 4.53.
- The width of the flood crossing bridge shall secure the flood crossing width. The passage structure shall withstand its concrete weight, water loads during the flood and vehicle and sidewalk loadings at the normal time. See Figure 4.54.
- After filling boulders in the connection between the slab of flood crossing bridge and ground surface, milk cement is poured into the gap of the boulders to further enhance the connection. See Photo 4.20.

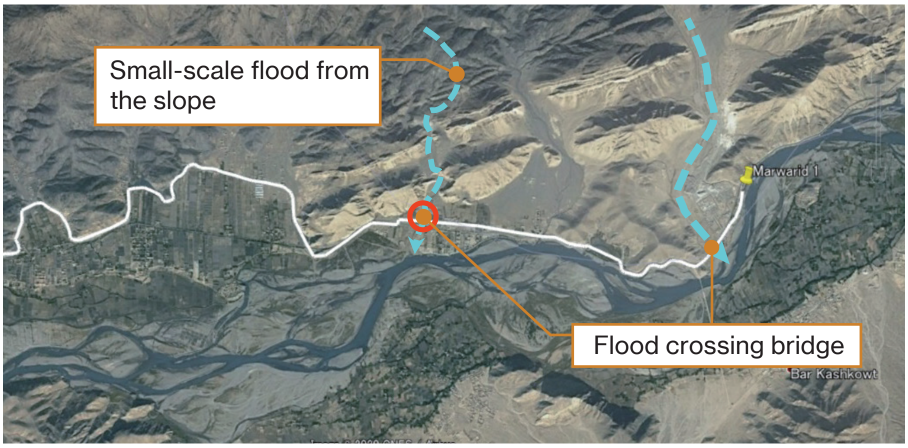
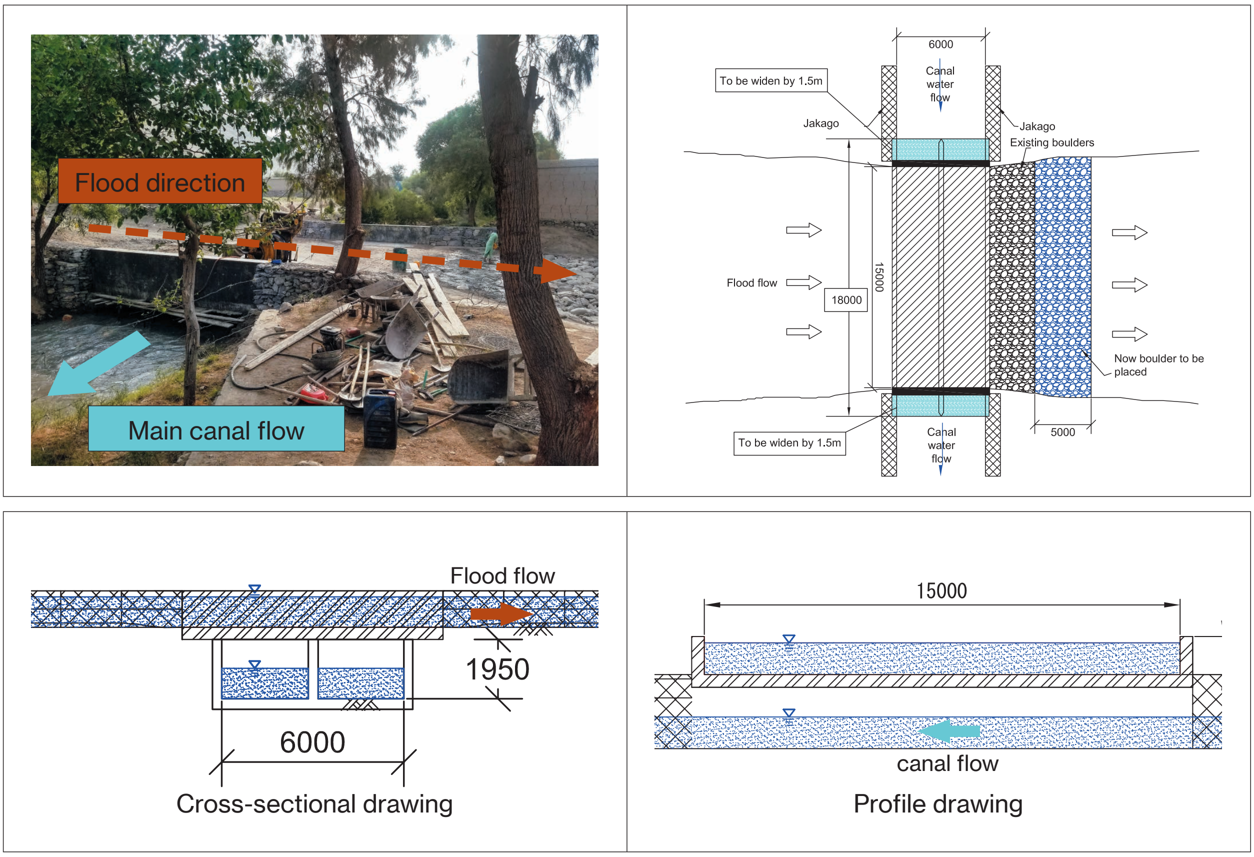


Figure 4.53 Flood Crossing Bridge Installation in the Flood Crossing Point [2)]



Note: The width of the flood crossing bridge was originally planned to be 30m or more, but due to land acquisition issues, it could only be widened to 18m.

Figure 4.54 Photo and Drawing of the Flood Crossing Bridge [2)]

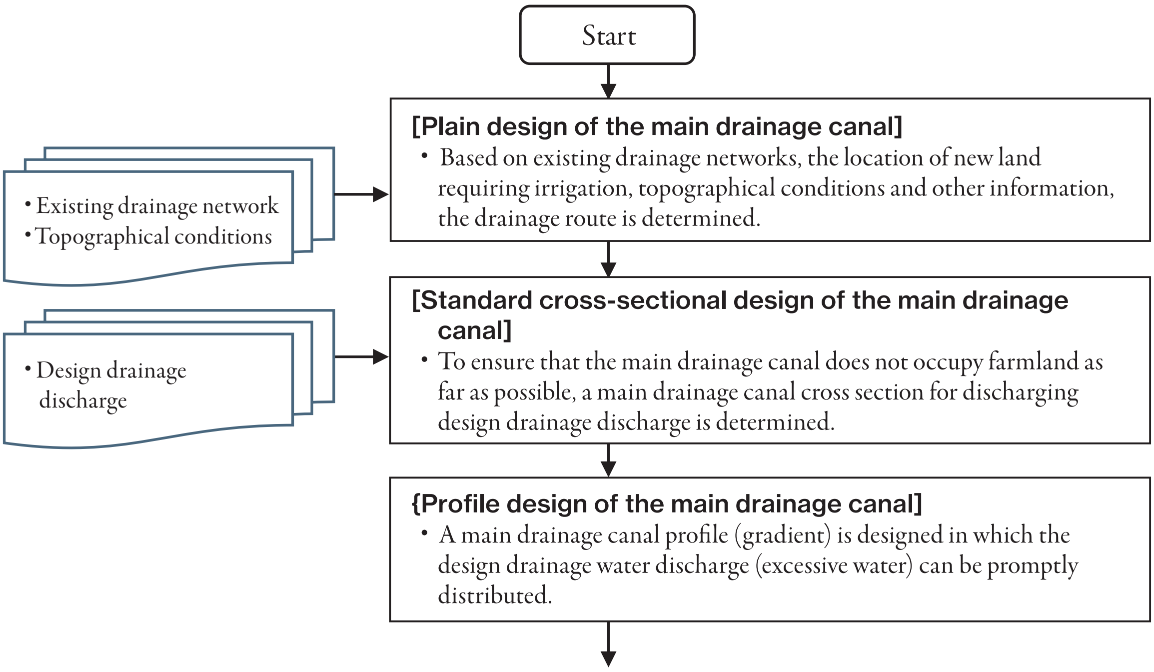
Photo 4.20 Connection Point of the Slab of the Flood Crossing Bridge to the Ground [1)]

4.6 Design of the Main Drainage Canal

4.6.1 | Basic Policy for Designing the Main Drainage Canal

When designing the main drainage canal, the priorities are shown as follows: The procedures for designing the main drainage canal are shown as the design process in Figure 4.55.

- Although the draining method includes gravity (natural) draining and machine draining using a pump, the former is applied in this drainage canal with the aspect of construction costs, maintenance, and management in mind.
- To promote drainage without occupying surrounding farmland as far as possible, the main drainage canal shall be the excavated method with a proper canal width, as a general rule. Adopting a main drainage canal method involving a dike embankment shall be avoided.



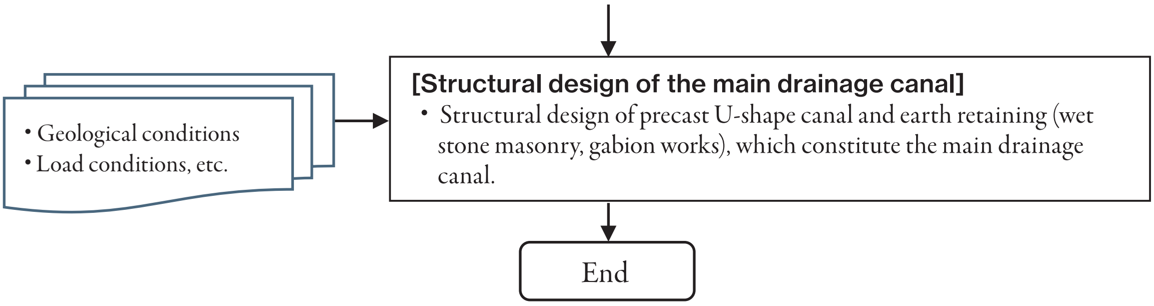


Figure 4.55 Process for Designing the Main Drainage Canal [2)]

4.6.2 | Type and Design Policy for the Main Drainage Canal

(1) Main Drainage Canal Type

The main drainage canal is planned as a drainage block in the irrigation beneficiary area connecting with the drainage destination river. The main drainage canal is composed of gabions, wicker works and canal bed lining like the main irrigation canal, but for the purpose of quicker drainage, a U-shaped canal is applied. The U-shaped ditch can be installed by crane hanging and precast concrete canal is adopted considering the efficient workability. The main drainage canal is smaller scale than the main irrigation canal and its establishment using precast materials offers superior workability and economic efficiency. In addition, since the U-shaped ditch can be constructed in Afghanistan, local residents are fully capable of maintaining and managing the facility.

(2) Design Policy of the Main Drainage Canal

The design specifications of the main drainage canal in the existing PMS irrigation project are shown below. Given the actual result and executability in Afghanistan, the following canal structure shall be designed. Meanwhile, the cross-section and gradient of the main drainage canal are designed in accordance with regional conditions, such as the topographical features and design drainage discharge, while referencing the following actual specifications from past projects. Figure 4.56 shows a cross-sectional drawing of the U-shaped main drainage canal.

- Canal structure of main drainage canal:	Depending on the site condition, appropriate works such as U-shaped canal, stone masonry, gabion works and wicker works should be applied.
- Cross-section of main drainage canal:	Upper width of main drainage canal: around 2.0 m (in case of precast U-shaped canal) Bottom width of main drainage canal: around 1.2 m (in case of precast U-shaped canal) Depth of main drainage canal: around 2.0 m (in case of precast U-shaped canal)
- Profile of main drainage canal:	I=0.0015 to 0.0040 (1/667 to 1/250)

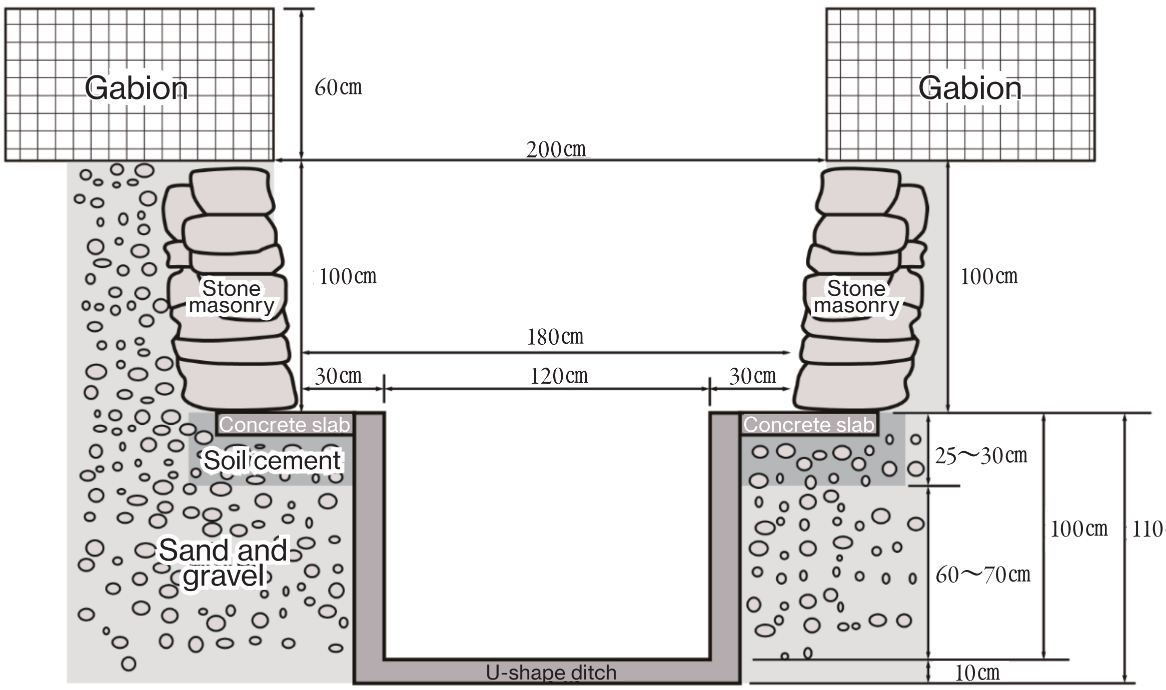


Figure 4.56 Cross-sectional Drawing of U-shaped Main Drainage Canal [3)]

4.6.3 | Specification Design of the Main Drainage Canal

(1) Plain Design of the Main Drainage Canal

The main drainage canal has the role to drain excessive water and rainwater away from the irrigation beneficiary area promptly. To streamline the main drainage canal, sufficient head shall be secured, considering the design flood water level of the river into which the excessive water drains out. The plain design of the main drainage canal shall be as follows:

- In irrigation beneficiary areas, the existing drainage canal system has more or less been formed based on the existence of the ancient storage area in accordance with the topographical/geographical and social conditions of the region. Accordingly, the regional drainage canal networks are surveyed to understand the existing drainage block composition regarded as the basis for the main drainage canal design.
- Drainage/overflow situations and wetland status in irrigation beneficiary areas are surveyed, when the wetland proportion is outstanding, the main drainage canal outlet to the river is relocated downstream side to secure the drainage head and consider a prompter drainage arrangement.
- The drainage canal from the new irrigation beneficiary area is connected to the main drainage canal at the nearest point at the lowest elevation in the drainage block. When connecting, it shall confirm that the elevation of the drain outlet in the new drainage block is higher than that of the existing drainage canal to be connected. Moreover, the drain outlet of the new drainage block is located to ensure it is not blocked by sediment and drift sand. Figure 4.57 shows an image of the connection between the new drainage block and the existing drainage block.
- An efficient drainage network is formed which allows water to be accumulated and conveyed via the main drainage canal by collecting the water drained from each drainage block in irrigation beneficiary areas by the regional drainage canal networks.
- The main drainage canal route is linear or at least gently curved, which allows to drain safely and efficiently.

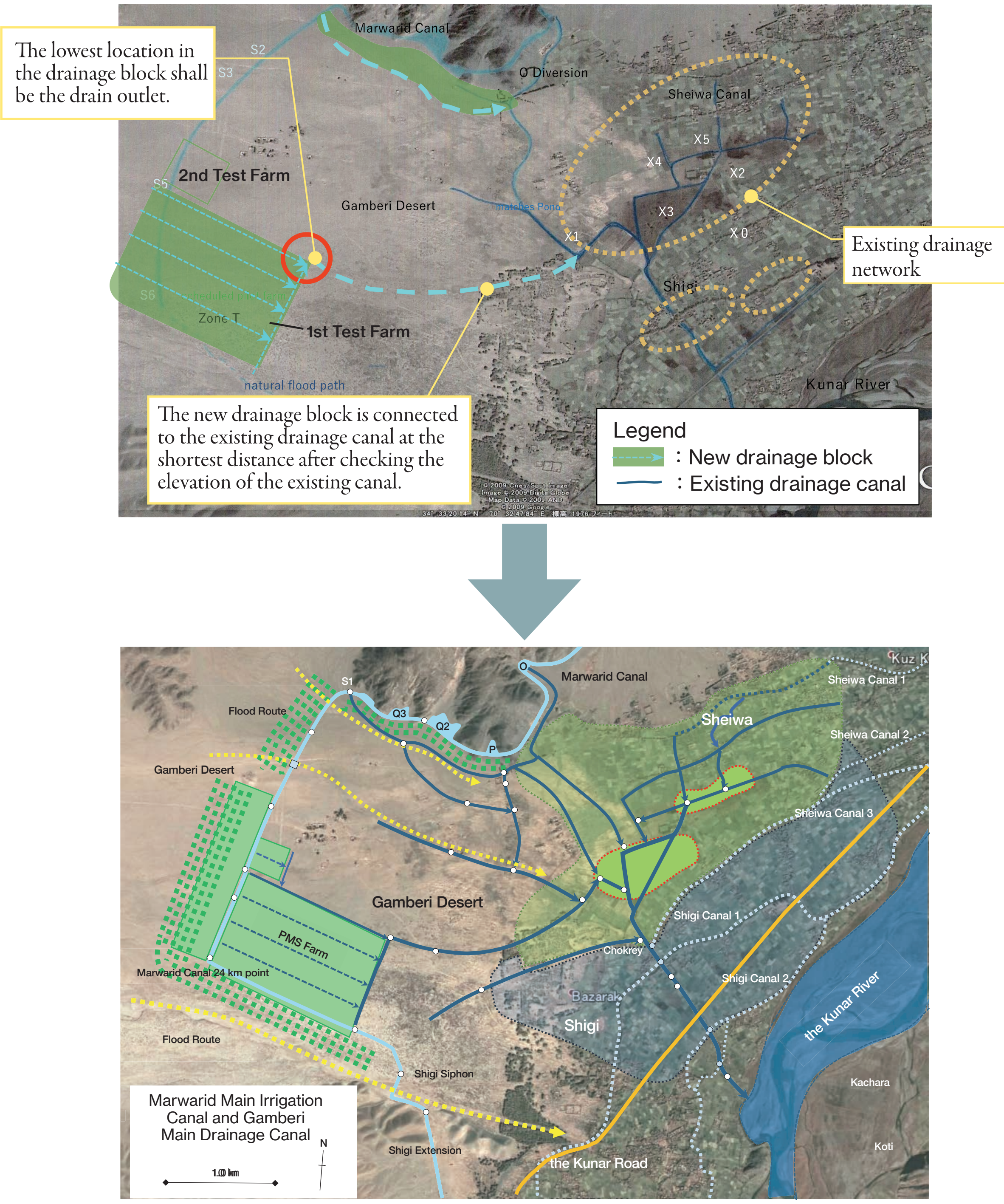


Figure 4.57 Example Connection between the New Drainage Block and Existing Drainage Network (Marwarid Weir)[3), 4)]

(2) Standard Cross-sectional and Profile Design of the Main Drainage Canal

The vertical gradient and standard cross-section of the main drainage canal are secured, which allow the design drainage water to be discharged. The profile design and standard cross-sectional design of the main drainage canal shall be as follows:

- The design drainage discharge is calculated as the value adding the rainwater drainage discharge to excessive irrigation water required from the intake water amount. However, given extremely low calculation accuracy, it is determined by the actual drainage measured discharge, as a general rule.

- The standard cross-section of the main drainage canal shall allow for design drainage discharge. Since a wider main drainage canal squeezes precious farmland, the main drainage canal shall not be excessively wide.
- The cross-section shall have a sufficient capacity for the design drainage discharge.
- The main drainage canal capacity is calculated using the Manning equation. The proper value of roughness coefficient shall be set depending on the structure of the drainage canal.
- At the crossing point of the main drainage canal with the main irrigation canal and a road, a drainage culvert is provided as part of the main drainage canal section.

(3) Structural Design of the Main Drainage Canal (in case of precast U-shaped canal)

The standard cross-section of the U-shaped main drainage canal is shown in Figure 4.56. The structural design of the main drainage canal shall be as follows:

- According to the past record of existing PMS irrigation project, considering the crane hanging and workability for installation, a precast concrete U-shaped canal, specifying approximately 660 kg weight, 120 cm width and 100 cm height constitutes the standard U-shaped canal. However, according to the design drainage discharge, proper canal materials corresponding to the site condition shall be applied. The production is relatively straightforward, as it is manufactured at a PMS workshop on the Gamberi farm.
- The foundation of the U-shaped canal is formed by replacing soft ground with gravel.
- The main drainage canal is formed by multiple cross-sections comprising precast concrete U-shaped canals, set stone masonry and gabion revetment. Since each structure is separated, the back of the structure shall be backfilled and compacted carefully without any looseness.
- Since the U-shaped canal in the lowest part receives upper wet stone masonry and gabion revetment loading and back-earth pressure, it shall secure a reinforced concrete member thickness and bar arrangement structure capable of withstanding this loading. Photo 4.21 shows the situation of U-shaped precast concrete construction.
- Given the wet stone masonry piled up at a steep gradient of 1:0.1 and upper gabion loadings, the stone masonry shall be a wet stone masonry structure using cement to secure revetment stability. The height of the gabion works at the top part is adjusted according to rear ground elevation. Photo 4.22 shows the upper part earth retaining wall of the main drainage canal.

Photo 4.21 Laying the Precast Concrete U-shaped canal [1)]

Photo 4.22 Upper Earth Retaining Wall of the Main Drainage Canal [1)]

(4) Stone Masonry Drainage Canal

Stone materials are used for the internal wall of the main irrigation canal to fill inside the gabion but also to protect the internal wall slope of a small canal in the regional drainage canal networks, in which the lower steps are covered thickly in boulders over 30 cm, larger than the gabion filling materials, as they withstand the earth pressure while square stones smaller in diameter are piled up in the upper wall as well as applying wicker works as shown in Figure 4.58. The example of stone masonry for a small canal wall is shown in Photo 4.23.

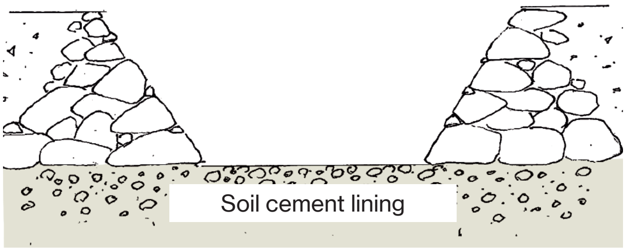


Figure 4.58 Stone Masonry for a Small Canal Wall[3)]

Photo 4.23 Example Stone Masonry for a Small Canal Wall [3)]