6-4-4 Improvement Plan of Irrigation Network System

The Project area is composed of two (2) areas largely. One area is construction area includes inflow and outflow facilities, the other one is irrigation filed which is managed by 4 WUAs. Herein after, the improvement plan is designed to following two areas respectively.

- **Target area 1 : Yeghvard reservoir and related facilities**
- **Target area 2 : Irrigation filed (composed by 4 WUAs)**

As for Target area 1, the reservoir condition shall deeply impact to design of related facilities. The basic conditions of Yeghvard reservoir are shown in Table 6-4-4.1.

Table 6-4-4.1 Basic Design Conditions of Reservoir

Full water level	EL 1,305.00m
Dam crest level	EL 1,307.55m
Dam Bottom level	EL 1,290.00m

6-4-4-1 Improvement Plan for Target Area 1

Stored water of 94MCM in Yeghvard reservoir is distributed to target irrigation area by the suitable conveyance facilities. Essential conveyance facilities are planned as shown in Figure 6-4-4.2 by following consideration and examination.

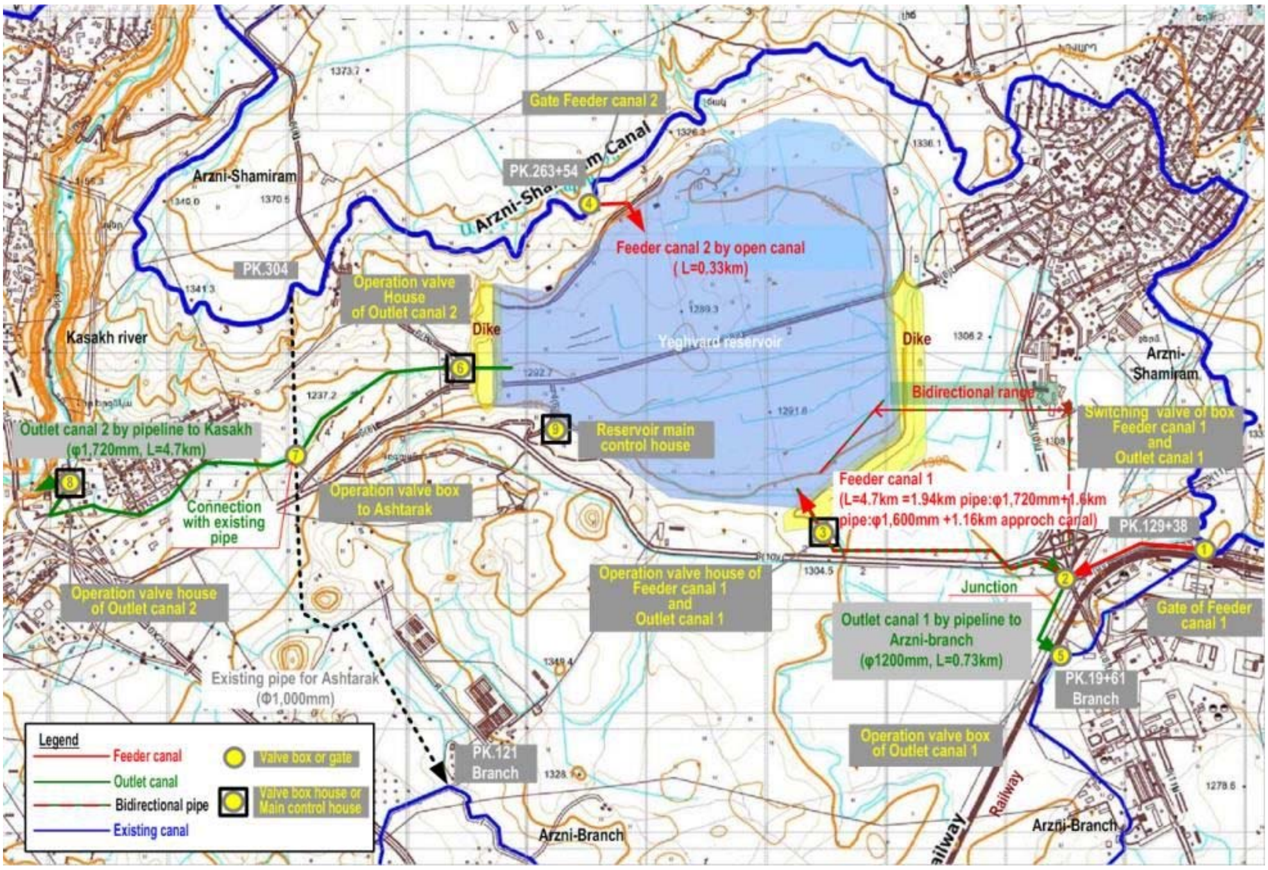
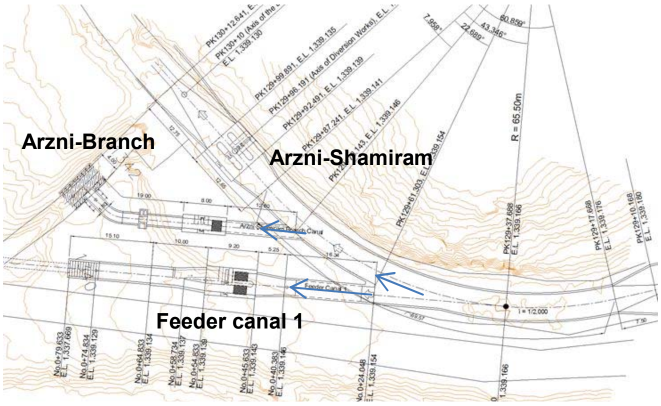
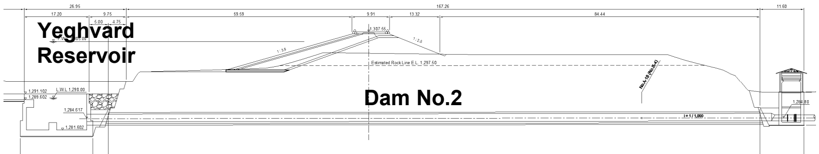


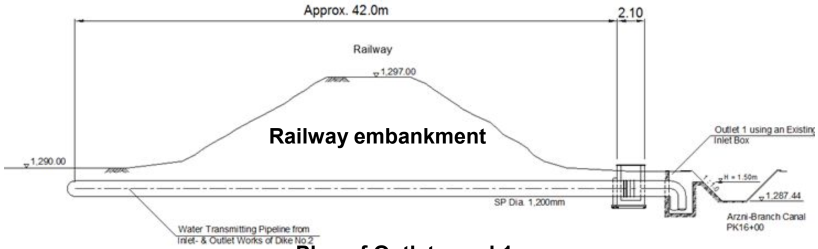
Figure 6-4-4.1 General Layout of Feeder and Outlet Canal for Targeted Area 1

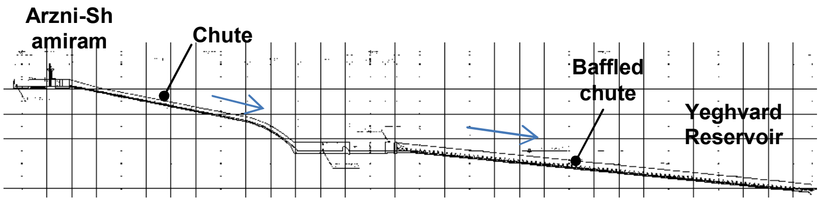
<table>
<tr><th>Canal</th><th>Typical section</th></tr>
<tr><td>Feeder canal 1
and Outlet canal 1</td><td>


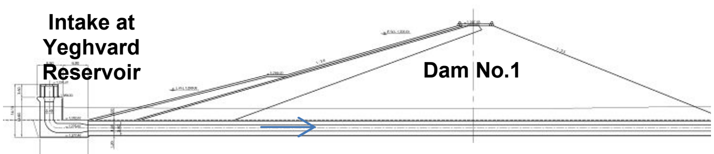
Inlet plan of feeder canal1



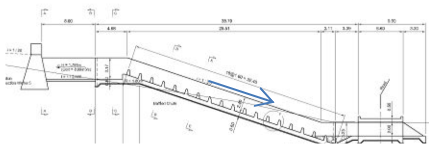
Approach canal of Feeder canal 1



Plan of Outlet canal 1
</td></tr>
<tr><td>Feeder canal 2</td><td>


Plan of Feeder canal 2
</td></tr>
<tr><td>Outlet canal 2</td><td>


Plan of Outlet Canal 2 at Dike 1


Plan of dissipater for Kasakh river (1/2)

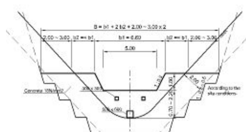

Plan of dissipater for Kasakh river (2/2)
</td></tr>
</table>

Figure 6-4-4.2 Typical Section of Planned Feeder and Outlet Canal

(1) Arrangement of inflow aspects to Yeghvard reservoir

a) Actual condition of inflow in Arzni-Shamiram canal

The Yeghvard reservoir is planned to 94MCM as design entire water volume. The reservoir is filled by the melted snow water in the limited season, **1st March to end of May**, through existent Arzni-Shamiram canal.

According to water allocation in a year, Arzni-Shamiram canal have been allowed to the following water allocation from March to May. The reservoir shall be taken into account the existent allocation schedule and the necessary conveyance way of water so as not to miss available water in targeted season. To achieve full store of 94MCM for designated 3 month only, **inflow 22.0m^3/s as maximum shall be made the most of availability and Arzni-Shamiram canal shall be enable to convey 22.0m^3/s without the problem.**

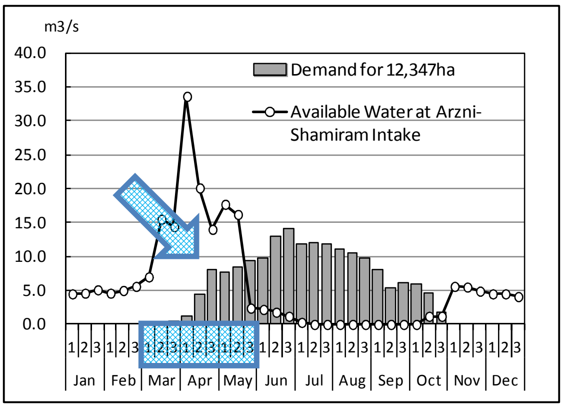


Figure 6-4-4.3 Available Water of Arzni-Shamiram in Year

Table 6-4-4.2 Water Allocation in Available Season in Arzni-Shamiram Canal

Month	March			April			May		
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
Vol. (m^3/s)	7.00	15.50	14.40	22.00	19.20	11.20	14.90	12.40	1.50

b) Hydraulic capacity of Arzni-Shamiram canal

In order to design the maximum utilization of Arzni-Shamiram canal, the hydraulic capacity should be verified. Given the designed reservoir location, the inflow canal (herein after "Feeder canal") to reservoir shall be planned at downstream of approx. PK130 where the beginning of Arzni- Branch is connected. PK130 is at distance of 13,000m from the beginning of Arzni-Shamiram canal.

Existent Arzni-Shmiram canal at downstream from PK130 is obtained and observed as following condition. The original design discharge is confirmed by "rehabilitation drawings by World Bank and Millennium Challenge Cooperation in Armenia (2011-2013). The actual discharge is assumed by visual observation of water trace on canal wall.

Table 6-4-4.3 Hydraulic Capacity Design and Actual

Location	Original design discharge (m^3/sec)	Expected discharge (m^3/s)	Remarks
PK 0+00 - PK 94+26	28.2	15.626	by OP. canal
PK 94+26 - PK 115+30	26.0	15.626	by OP. canal
PK 115+30 - PK 130+17	24.0	15.626	by OP. canal
PK 130+17 - PK 181+18	17.6	15.0	at east of planned reservoir / by OP. canal
PK 181+18 - PK 311+60	16.8	15.0	From PK181+18 to PK190+35 at Yeghvard city by box culvert
PK 311+60 - PK 350+95	15.0	15.0	at north and west of planned reservoir / by OP. canal

Note) Hatch show at range of D.S. of PK130

As a result of the careful consideration of hydraulic capacity, the expected maximum discharge is assumed to 15m^3/s. However, considering to safety pass of flow and to avoid overflow at narrow and shortage freeboard section, the maximum design discharge at downstream from PK130 shall be 13m^3/s. Because the enhancement of the canal capacity in Yeghvard city, where the box culvert has been applied as conveyance structure, is raised social difficulty and issue due to the residence area.

On the other hand, Arzni-Shamiram canal at upstream for PK130 which is constructed by open canal, is observed shortage capacity section such as short freeboard to convey 22m^3/s of maximum. Although Arzni-Shamiram canal was rehabilitated by World Bank project and by the program of Millennium Challenge Cooperation in Armenia, the rehabilitation could not be completed in the program because of lack of design. To satisfy the target maximum discharge of 22m^3/s, Arzni-Shamiram canal at upstream for PK130 shall be rehabilitated to secure the necessary freeboard by raising wall at distance of PK20 to PK45, PK70-PK90 and PK95 to PK105.

The available hydraulic capacity and necessary rehabilitation works in Arzni-Shamiram canal should be designed as Table 6-4-4.4;

Table 6-4-4.4 Hydraulic Condition for Allowable Capacity in Arzni-Shamiram Canal

Location	Max. discharge (m^3/s)	Remarks
PK0+00 ~ PK130+17	**22.0**	Leakage protection and other rehabilitation L=5.5km (PK20 to PK45, PK70 – PK90 and PK95-PK105)
Downstream of PK130+17	**13.0**	

Note) PK130+17 is at connected of Arzni-Branch

(2) Plan of feeder canals

a) Necessary feeder canals

As the result of the necessity of maximized use of discharge and consideration of hydraulic condition in Arzni-Shamiram canal, to reserve 94MCM for the designated 3 months, necessary feeder canals are planned by following options. As for the basic concept of alignment for canal, it is important that to avoid negative effect to land use and unnecessary long alignment. Therefore, structure shall be planned along road area and farrow area basically. With careful consideration of land-use and topographic condition, alignment of feeder canals and each option are outlined as followings;

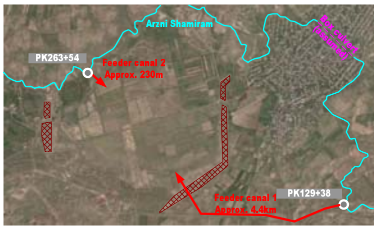
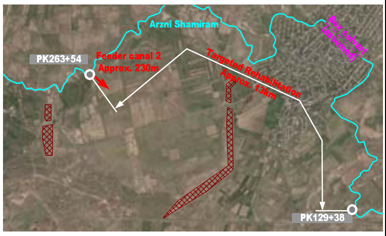
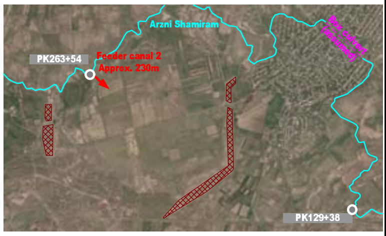
- ✓ **Option 1**: **Two feeder canals**, Feeder canal 1 and Feeder canal 2, convey water of max. 22m^3/s to reservoir.
  Feeder canal 1 should be from PK129+196 of Arzni-Shamiram canal to at south of reservoir along public road by pipeline and Feeder canal 2 should be at around PK263+20 of Arzni-Shamiram canal by open canal, which is at north of reservoir.
- ✓ **Option 2**: **One feeder canal** same as Feeder 2 of Option 1, convey water of max. 22m^3/s to reservoir. **The upgrading of Arzni-Shamiram canal** from PK129+196 to around PK263+20, L= approx. 13km, shall be essential to pass 22m^3/s because of short hydraulic capacity at that section.
- ✓ **Option 3**: **One feeder canal** same as Feeder 2 of Option 1 is planned, but conveyance water should be 13m^3/s so as not to be over allowable capacity in Arzni-Shamiram canal. To reserve 94MCM in reservoir, **periods of inflow from Arzni-Shamiram canal shall be extended to 4 - 5 months** with approval by related agency.

Considering of below comparison, **Option 1 is applied plan for feeder canal. Two feeder canals shall be planned.**

Table 6-4-4.5 Intake Allocation of Feeder Canal

Option	Facility	Rough location of facility	Max. design discharge (m3/s)	Intake vol. (m3/s)	Remarks
Option 1	Feeder canal 1	PK129+196 (near B.P of Arzni-branch)	**9.0**	22.0	Conveyance by pipeline to reservoir
	Feeder canal 2	PK263+20 (outside of Yeghvard city)	**13.0**		1) Conveyance by OP. canal to reservoir 2) Discharge is allowed to pass in Yeghvard city due to below capacity.
Option 2	Feeder canal 2	PK263+20 (outside of Yeghvard city)	22.0	22.0	1) Conveyance by OP. canal to reservoir 2) Upgrading of Arzni-Shamiram L=13km
Option 3	Feeder canal 2	PK263+20 (outside of Yeghvard city)	13.0	13.0	1) Conveyance by OP. canal to reservoir 2) Extension of available inflow periods

Table 6-4-4.6 Comparison of Feeder Canal Plan

Item	Option 1	Option 2	Option 3
Outline	 ■Plan is Two feeder canals ■Feeder canal 1 (max 9m^3/s) is at PK129+196 by pipeline, Feeder canal 2 (max 13m^3/s) is at north of reservoir by OP. canal. ■Max. intake volume is 22m^3/s	 ■Plan is one feeder canal with upgrading Arzni-shamiram canal ■Feeder canal is at north of reservoir by OP. canal. ■Max. intake volume is 22m^3/s	 ■Plan is one feeder canal without upgrading Arzni-shamiram canal ■Extension of available inflow periods to 4~5 months ■Feeder canal is at north of reservoir by OP. canal. ■Max. intake volume is 13m^3/s
Hydraulic feature	■Feeder canal 1(F.C.1) by pipeline has dully effective water head (Δhe) to convey water. Δhe =approx. 50m > 43m =Δhf ■Feeder canal 2(F.C.2) by OP. canal is dully applied, because B.P. of F.C.2 is far higher than F.W.L. of reservoir EL 1333.8m > F.W.L.1,305m ⇒**Hydraulic conditions are solved** **(+)**	■F.C.2 by OP. canal is dully applied, because B.P. of F.C.2 is far higher than F.W.L. of reservoir. EL 1331.1m > F.W.L.1,305m ■Upgrading of Arzni-Shamiram canal for 13km from PK129+196 is necessary of raising wall by approx. 1m to pass 22m^3/s include Yeghvard city. ⇒**Hydraulic conditions are solved** **(+)**	■F.C.2 by OP. canal is dully applied, because B.P. of F.C.2 is far higher than F.W.L. of reservoir. EL 1331.1m > F.W.L.1,305m ■Upgrading of Arzni-Shamiram canal for 2.7km is needless. ■But, longer inflow periods than other option is necessity. ⇒ **Hydraulic conditions are no obstruction** **(+)**
Construction feature	■Construction site of new facilities is two. ■F.C.1 works should be arranged road traffic and required to approval of road works. ⇒**Easy construction works** **(+)**	■Construction site of new facilities is one. ■Box culvert at Yeghvard city should be required to the upgrading, but land-use permission and consensus are essential. Construction works shall be avoided big negative effect to residents and case to costly works. ⇒**Difficult construction works and negative effect to Yeghvard city.** **(-)**	■Construction site of new facilities is one. ⇒**Easiest construction works** **(+ +)**
Social issue	■Arranged road traffic and permission of road works are required, but, those would be obtained by usual procedure. **(+, -)**	■Land-use permission and consensus are essential. Complexed social impact arise. **(- -)**	■Arrangement of permission on inflow for 4 - 5 months periods is essential, but it is negative prospection, due to difficulty of changing of water allocation on Hrazdan river. **(- -)**
Assess-ment	Total merit and demerit is 2+ (Applied)	Total merit and demerit is 2-	Total merit and demerit is 1+

b) Outline of Feeder canal 1 (at east of reservoir)

Feeder canal 1 is located from PK130+38 of Arzni- Shamiram canal to at south of reservoir along public road by pipeline. The reason of pipeline is as followings;

- ✓ To store the water up to F.W.L.1,305m, the water head shall not less than 15m of effective water head which is difference between EL.1,290m at B.P(Beginning Point) of Feeder canal 1 and F.W.L1,305m,
- ✓ OP. canal cannot keep within above effective water head on conveyance because water head are forced to affect by topographic level on its way which is descending toward reservoir. OP. canal

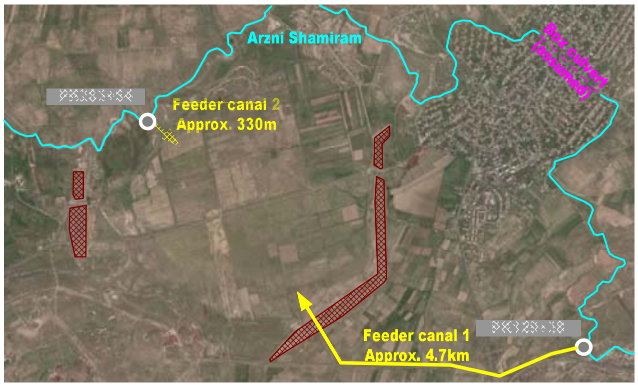
would be installed pump system to cross the reservoir dike, if applied,

- ✓ In the contrast to OP. canal, pipeline is only affected between topographic level at B.P. and E.P. of pipe, loss of pipe length and partial head-loss which means that the conveyance is enable to pass under the dike without pump,
- ✓ In addition, pipeline affect less negative impact on existent land-use along the alignment because of underground structure against OP. canal,
- ✓ OP. canal shall permanently need 3m width and 2m depth of section for same discharge of pipe and add approx. 4m maintenance road at side which means that the affected area by construction would be impacted on existent farm land,
- ✓ Pipeline, therefore, shall be applied by technical and social impact consideration

Basic layout and hydraulic conditions is shown on Table 6-4-4.7.

Table 6-4-4.7 Basic Layout and Maximum and Minimum Discharge of Feeder Canal 1

Discharge (m^3/s)		Month
Maximum	9.0	2nd period in Mar. to 2nd period in May
Minimum	1.11	3rd period in May
Length of canal	L=4.7km by pipeline	



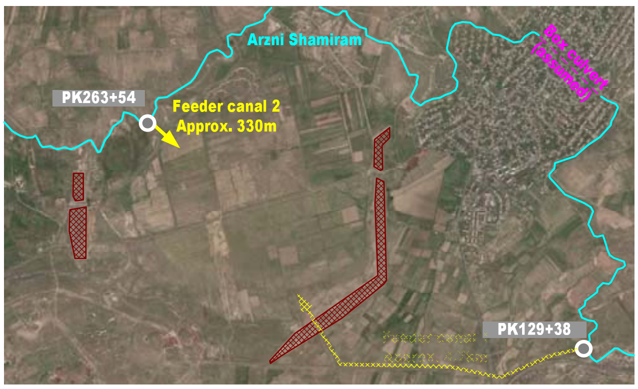
c) Outline of Feeder canal 2 (at north of reservoir)

Feeder canal 2 is located from PK263+54 of Arzni-Shamiram canal to at north of reservoir which is the closest to reservoir, no influence by roughness land-form and no negative impact on farm-land by construction. In addition, as the location is observed at considerable high place, canal alignment is designed by mountainous slope with OP. canal.

By approx. 26.5m of effective water head between approx. W.L.1,333.45m of Arzni-Shamiram canal of PK263+54 and F.W.L.1,305m, is available for OP canal, but the suitable dissipater shall be applied to take measure for high velocity.

Table 6-4-4.8 Basic Layout and Maximum and Minimum Discharge of Feeder Canal 2

Discharge (m^3/s)		Month
Maximum	13.0	2nd period in Mar. to 2nd period in May
Minimum	2.2	3rd period in May
Length of canal	L=0.33km by OP. canal	



Basic layout and hydraulic conditions is shown on Table 6-4-4.8.

(2) Plan of outlet canals

The outflow from reservoir (herein after "outlet canal") is toward three areas pursuant to expected water operation. The outlet canal should be planned to effective connect with existent canals. In addition, the alignment should be designed to avoid large impact and unnecessary long alignment.

Table 6-4-4.9 Target WUAs of Outlet Canals

Targeted area	Method of conveyance to area
Yeghvard WUA	Outlet canal 1 : From reservoir to Arzni-branch
Ashtarak WUA	Outlet canal 2 : From reservoir to existent pipe which convey water to Arzni-Branch at PK121 of Arzni-Branch
Khoy and Vagharshapat WUA	Outlet canal 2 : From reservoir to Kasakh river, which way convey water to targeted area through Kasakh river

a) Outline of Outlet canal 1 (From reservoir toward Yeghvard WUA)

Yeghvard WUA is irrigated from Arzni-Branch canal. The Outlet canal 1 should be connected to Arzni-branch canal from south of reservoir, which junction should be at upstream of Arzni-Branch before Yeghvard WUA's distributer. The situation of south area of reservoir is observed national road and wide farmland. The alignment should be planned along of road to avoid impact on farmland.

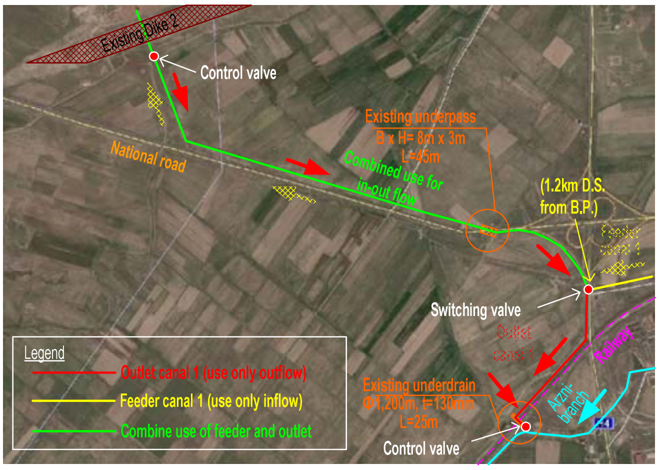


Figure 6-4-4-4 Alignment of Outlet Canal 1

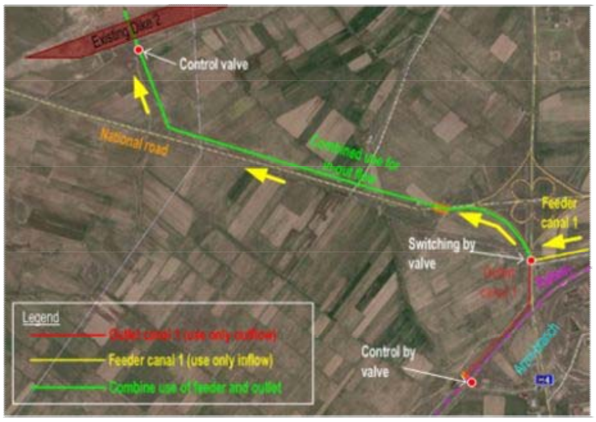
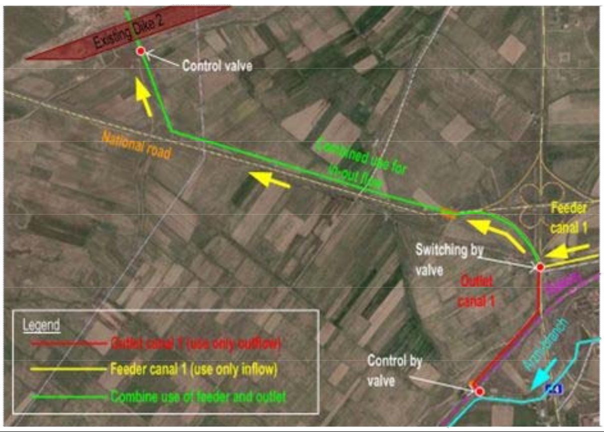
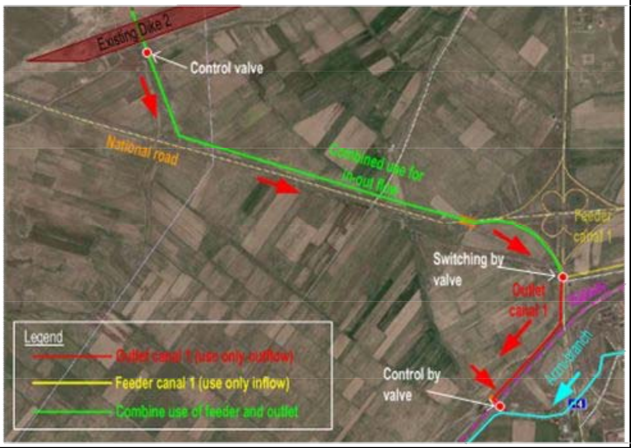
The alignment of Outlet canal 1 is almost same as Feeder canal 1 due to topographic condition. Although the alignment could be set in parallel of Outlet canal 1 and Feeder canal 1, which means that individual pipes are installed, the cost for two (2) pipes construction would be increased.

To save the cost, the Outlet canal 1 should be connected at Feeder canal 1 of approx. 1.2km from B.P. of Feeder canal 1 (refer to right figure). Hence, **from downstream 1.2km of Feeder canal 1, its function of pipes should be shared with Feeder canal 1 and Outlet canal 1 i.e. be combined with in-flow and out-flow.**

To operate these pipes, three valves are required at junction of two pipes as switching, at shortly downstream of the dike and at end of Outlet canal 1.

According to the water allocation of Feeder canal 1 and Outlet canal 1, operation should be done as Table 6-4-4.10. The opposite direction of water flow does not raise at junction of Feeder canal 1 and Outlet canal 1.

Table 6-4-4.10 Operation of Feeder Canal 1 and Outlet Canal 1 by Water Allocation

<table>
<tr><th rowspan="2">Month</th><th rowspan="2">Period</th><th colspan="3">Canal</th><th rowspan="2">Direction of water flow F.C.1 and O.C.1</th></tr>
<tr><th>F.C.1 note)</th><th>O.C.1 note)</th><th>Arz-Br.</th></tr>
<tr><td rowspan="3">March</td><td>1st</td><td>7.00</td><td rowspan="8">no-operation by End of May</td><td rowspan="4">no-operation by April 2nd term</td><td rowspan="4">
</td></tr>
<tr><td>2nd</td><td>9.00</td></tr>
<tr><td>3rd</td><td>9.00</td></tr>
<tr><td rowspan="3">April</td><td>1st</td><td>9.00</td></tr>
<tr><td>2nd</td><td>8.72</td><td>0.28</td><td rowspan="5">
</td></tr>
<tr><td>3rd</td><td>8.84</td><td>1.16</td></tr>
<tr><td rowspan="3">May</td><td>1st</td><td>7.67</td><td>1.33</td></tr>
<tr><td>2nd</td><td>8.00</td><td>1.00</td></tr>
<tr><td>3rd</td><td>1.11</td><td>0.39</td></tr>
<tr><td rowspan="3">June</td><td>1st</td><td rowspan="3">no-operation by next March</td><td>0.50</td><td rowspan="3">Irrigation water is covered by Outlet canal 1</td><td rowspan="3">
</td></tr>
<tr><td>2nd</td><td>1.66</td></tr>
<tr><td>3rd</td><td>2.11</td></tr>
</table>

Note) F.C. 1 is Feeder canal 1, O.C.1 is Outlet canal 1

According to the site survey, Outlet canal 1 should be located along existent road to avoid the farmland, then, can be reached to Arzni-Branch canal by crossing the railway embankment.

The crossed point at the railway embankment is observed an existent concrete pipe, ϕ 1,000mm, in the embankment. Since diameter of existing pipe is ϕ 1,000mm. It could be available to pass the planned pipe under the embankment, but the ground level around railway embankment is almost same as reservoir LWL 1,290m. This means that effective water head would be shortage to reach

Arzni-Branch canal. Therefore, in the plan in this Survey, the tunnel by pipe thrusting method should be applied. The location of junction with Arzni-Branch canal is around PK.19+61, 1.9km from B.P, of Arzni-Branch canal.

The maximum and minimum discharge by the existent water allocation of Yeghvard WUA from Arzni-Shamiram canal is shown in Table 6-4-4.11.

Table 6-4-4.11 Maximum and Minimum Discharge of Outlet Canal 1

Discharge (m^3/s)		Month
Maximum	2.33	1st period in July
Minimum	0.22	3rd period in September
Length of canal	L=0.73km by pipeline (partially combined with Feeder canal 1)	

In case of OP. canal, in order to detour the high land area which is almost EL.1,300m and to reach to Arzni-Banch canal, the seasonal water stream shall be used as canal alignment (refer to Figure 6-4-4.5). Since topographic situation at south of reservoir is higher than reservoir bottom level of EL.1,290m, according to the site survey. The alignment would be meandered and be longed to 6.5km which is considerably longer than the pipeline. OP. canal should not be applied to this condition.

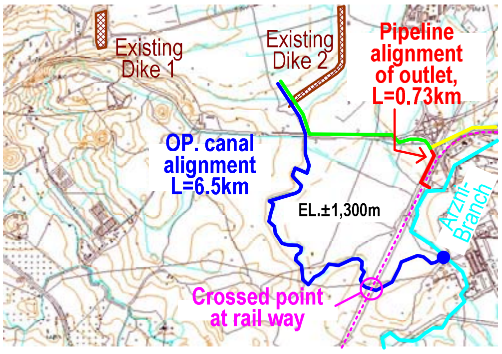


Figure 6-4-4.5 Alignment of OP. Canal

b) Outline of Outlet canal 2 (From reservoir to existent pipe and Kasakh river)

In water operation of Ashtarak WUA presently, the conveyance pipe line (ϕ 1,000mm) is connected directly to Arzni-Shamiram canal at around PK304 and reach to Arzni-Branch canal at around PK121 over the hill. In addition, Yeghvard reservoir need to convey the irrigation water to Khoy WUA and Vagharshapat WUA.

As for the conveyance to Ashtarak WUA, considering the present water operation, outlet canal should be divided around existing pipeline and connected to it. Since the existent pipeline is located at west of reservoir and crossed at valley topography with appearing on ground, the alignment of outlet canal should be used along valley without problem of land property.

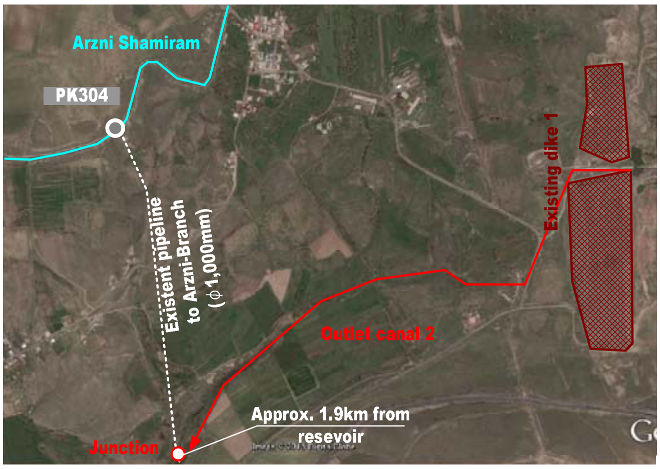


Figure 6-4-4.6 Alignment of Outlet Canal 2 till Existing Pipeline

According to the site survey, the existent pipe line to Arzni-Branch canal conveys the water over the hill which by using effective water head between Arzni-Shamiram canal (±WL.1,330m) and E.P. of pipeline at Arzni-Branch canal (±WL.1,276m). The level of hill on pipeline's way is EL1,270~1,280m.

On the other hand, the bottom level of reservoir is planned EL.1,290m, so effective water head is possible to approx.14m. Considering the loss water head for pipe length and some, 13.7 m in total loss include 10% extra loss for safety is calculated, which show to be allowable to apply above alignment of outlet canal. **In this situation, OP. canal cannot be applied naturally.**

The maximum and minimum discharge by the existent water allocation of Ashtarak WUA from Arzni-Shamiram is shown on Table 6-4-4.12.

Table 6-4-4.12 Maximum and Minimum Discharge of Outlet Canal 2 for Ashtarak WUA

Discharge (m^3/s)		Month
Maximum	0.56	1st period in July
Minimum	0.05	3rd period in September

As for the conveyance to Khoy WUA and Vagharshapat WUA, the original plan by USSR, which is to use Kasakh river as conveyance system, should be followed by new irrigation system. Presently, these WUAs are irrigated by Arzni-Shamiram canal, Lower Hrazdan canal, Kasakh Intake at Kasakh river and Pump stations of Aknalich and Metsamor.

Basically, the alignment of Outlet canal 2 should be extended from above connection with existing pipeline and reach to Kasakh River. According to the site survey, the valley topography is extended to Kasakh river through near the south area of village. Therefore, the alignment should be along the valley as well. At near Kasakh River of Outlet canal 2, however, the topographic condition is drastically changed, the suitable dissipater shall be planned to ease release to Kasakh river.

Figure 6-4-4.7 Outlet Canal 2 at near Kasakh River

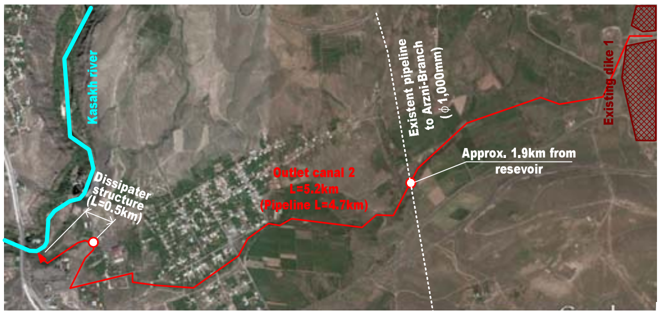


Figure 6-4-4.8 Alignment of Outlet Canal 2 till Kasakh River

As for the design from Outlet canal 2, since discharge is planned to release to Kasakh river, the influence for Kasakh river need to take it into account. Considering the hydraulic capacity of Kasakh river, the maximum discharge added Outlet canal 2 shall be within hydraulic capacity of Kasakh river. According to the record discharge for 30ys from 1983 to 2013 at Aparan dam observation, the average of every 10days discharge for each month are shown on table 6-4-4.13.

Table 6-4-4.13 Total Discharge in Usual between Outlet canal 2 and Kasakh River

Month	Jan. (m3/s)			Feb. (m3/s)			Mar. (m3/s)			Apr. (m3/s)			May (m3/s)			Jun. (m3/s)		
Kasakh	2.64	2.59	2.59	2.59	2.60	2.64	3.21	3.60	4.83	8.25	7.58	4.68	3.75	3.00	2.64	2.65	2.53	2.53
O.C. 2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.16	0.66	0.75	0.56	0.22	6.48	8.24	8.99
Total	**2.64**	**2.59**	**2.59**	**2.59**	**2.60**	**2.64**	**3.21**	**3.60**	**4.83**	**8.25**	**7.74**	**5.34**	**4.50**	**3.56**	**2.86**	**9.13**	**10.77**	**11.52**
Month	Jul (m3/s)			Aug. (m3/s)			Sep. (m3/s)			Oct. (m3/s)			Nov. (m3/s)			Dec. (m3/s)		
Kasakh	2.39	2.38	2.38	2.42	2.43	2.41	2.48	2.38	2.33	2.45	2.60	2.65	2.80	3.01	2.79	2.72	2.66	2.66
O.C. 2	6.61	6.88	6.75	5.95	5.69	5.13	3.37	2.39	3.32	3.02	0.59	0.25	0.0	0.0	0.0	0.0	0.0	0.0
Total	**9.00**	**9.26**	**9.13**	**8.37**	**8.12**	**7.54**	**5.85**	**4,77**	**5.65**	**5.47**	**3.19**	**2.90**	**2.80**	**3.01**	**2.79**	**2.72**	**2.66**	**2.66**

Note) O.C.2 is Outlet canal 2

According to water relation between the discharge record of Kasakh river for every month and necessary discharge from Outlet 2 which shown on table 6-4-4.13, total **usual discharge** of Outlet canal 2 and Kasakh river, need to be less than 11.52m^3/s.

However, Outlet canal 2 is not only responsible to convey the water to the targeted area, but also to perform the emergency discharge in unusual situation. In the assessment of record for 30ys on **maximum discharge** and site survey along Kasakh river which are shown on "2) Discharge volume from Yeghvard reservoir in Chapter 6-5-7 Basic Design of Related Facilities (Emergency Discharge Structure)", 13.7m^3/s is assumed as the allowable hydraulic capacity of at downstream of Kasakh river**.**

Therefore, given of 13.7m^3/s as allowable hydraulic capacity of Kasakh river, maximum discharge in emergency conditions should be ranged from max. 12.00m^3/s to 12.82m^3/s which are calculated by comparison between varying effective water head at reservoir and the allowable discharge of Kasakh for 13.7m^3/s (refer to Chapter 6-5-7, 2)). The design conditions of maximum and minimum discharge are shown on Table 6-4-4.14.

Table 6-4-4.14 Maximum and Minimum Discharge of Outlet Canal 2 for Khoy WUA and Metsamor WUA

Discharge (m^3/s)		Month
Maximum note)	1) Not less than max. 12.10~12.82 m^3/s for varying reservoir water level, but not more than max. 13.7m^3/s (for pipe design) 2) 13.7 m^3/s (for dissipater)	emergency conditions
Minimum	0.16	2nd period in April
Length of canal	L=4.7km by pipeline (except for section dissipater of 0.5km)	

Note) Definition of max. discharge is referred to "6-5-7 Emergency Discharge Structure"

Accordingly, design condition of Outlet canal 2 should be taken into consideration value of Table 6-4-4.12 and Table 6-4-4.14.

(3) Structural design of Feeder canal and Outlet canal

a) Feeder canal 1 and Outlet canal 1

Feeder canal 1 and Outlet canal 1 are planned by pipeline. Pipeline of Feeder canal is ϕ 1,600mm and Outlet canal 1 is ϕ 800mmm, which are applied by hydraulic calculation. According to the previous consideration, Feeder canal 1 and Outlet canal 1 are needed to share with each of function. In addition, these conveyance structures are designed by pipeline. The following conditions should be considered.

- ✓ In order to distribute the irrigation water from Arzni-Shamiam canal, new intakes for Feeder canal 1 and Arzni-Branch canal should be constructed for the suitable regulation. In addition, the reconstruction of existing regulator which is located at shortly downstream of planned area, should be planned due to frost damage.
- ✓ The distributed water promptly flow into the pipe and pass the underground until inside of reservoir. To pass Dam No.2, the conveyed water should be passed by tunnel under Dam No.2. Because foundation of structure must be avoided to construct on embankment like dike.
- ✓ For the Feeder canal 1 and Outlet canal 1, inlet/outlet in reservoir should be equipped with dissipated block which is stationed at inlet/outlet in order to alleviate high velocity in inflow situation. In addition, collecting channel should be designed so as to easy collect water from reservoir at situation of low water level.

Accordingly, main structures of Feeder canal 1 and Outlet canal 1 are as shown in Figure 6-4-4.9;

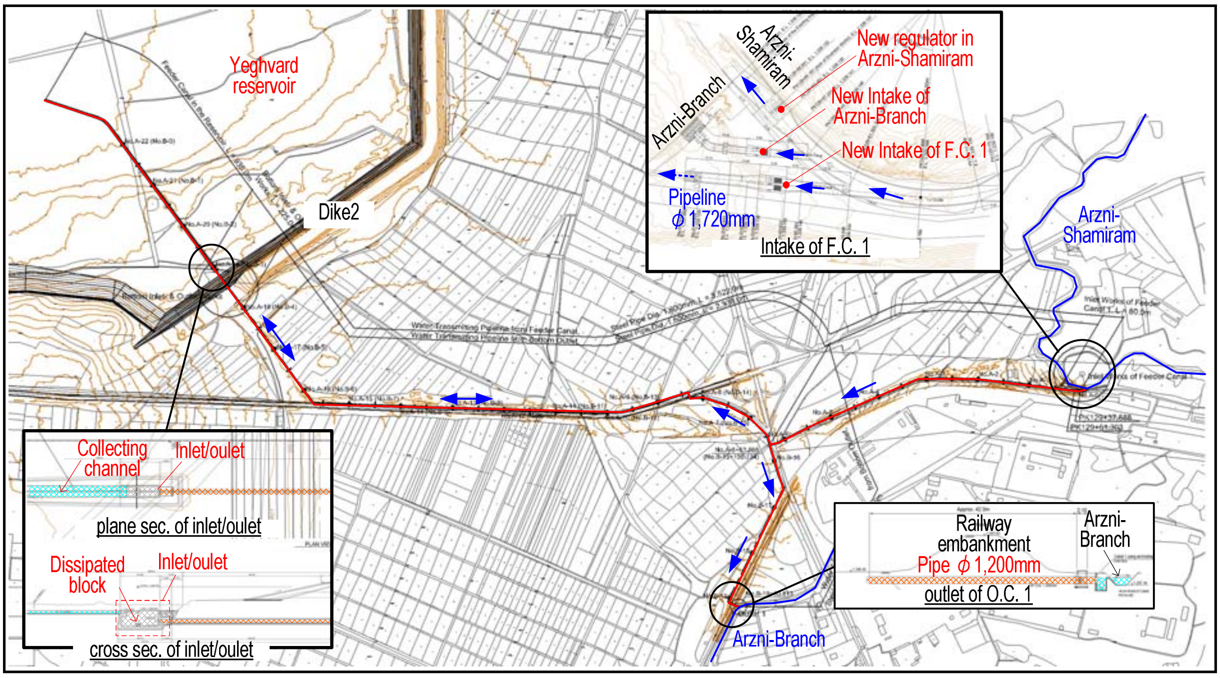


Figure 6-4-4.9 Plan of Feeder Canal 1 and Outlet Canal 1

b) Feeder canal 2

Feeder canal is planned by open canal. According to the topographic feature, inlet and outlet of feeder canal 2 has big difference of ground level which is almost 40m and its slope alignment is approx. 1/8. This means that flow velocity would be so high and it should be taken into consideration the effect of erosion to reservoir. Therefore the chute and baffled chute facilities are needed to secure the dissipated effect.

In addition, in order to regulate the inflow water to facility depend on seasonal demand, the regulator gate should be installed to Arzni-Shamiram canal and intake of Feeder canal 2.

Accordingly, main structures of Feeder canal 2 are as shown in Figure 6-4-4.10;

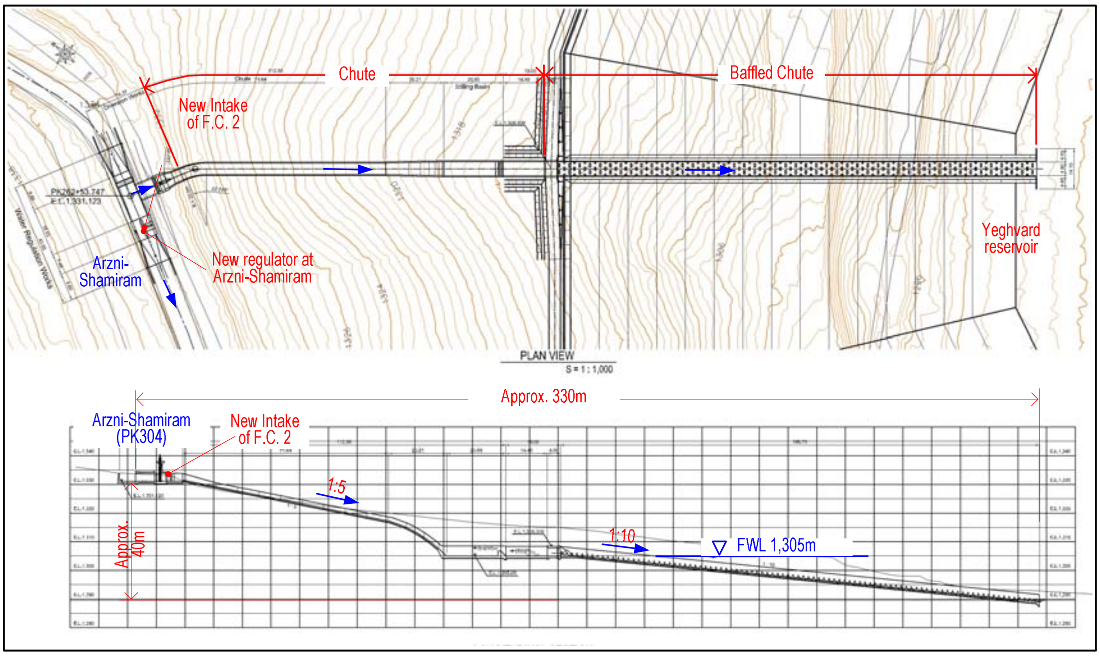


Figure 6-4-4.10 Plan of Feeder Canal 2

c) Outlet canal 2

Outlet 2 is planned by pipeline. Pipeline is ϕ 1,700mm, which are applied by hydraulic calculation. Outlet canal 2 needs to take into account as followings.

- ✓ In order to convey the irrigation water to Ashtarak WUA, Outlet canal 2 is needed to connect with existing pipe (ϕ 1,000mm) at distance of approx. 1.9km from reservoir.
- ✓ Function of Outlet canal 2 is to convey the usual irrigation water to farmland of Ashtarak WUA, Khoy WUA and Vagharshapat WUA, in addtion, to release the emergency discharge.
- ✓ Big water head difference between reservoir and outlet of pipeline which is almost 150 meter, would be affected to discharge control. Therefore the fixed corn valves should be installed at outlet to control the varying water demand depend on season. According to the yearly water demand, two type of valves, ϕ 1,200mm and ϕ 350mm should be installed to regulate ranged discharge.
- ✓ In addition, considering the topographic feature from outlet of pipeline to Kasakh river, suitable dissipated facility and structure should be constructed. According to careful consideration, five of protection walls like slit dam should be stationed at upstream and baffled chute should be stationed at downstream. Then finally the crossing box culvert to pass the existing road at near Kasakh river is constructed to approach Kasakh river.

Accordingly, main structures of Outlet canal 2 are as shown in Figure 6-4-4.11;

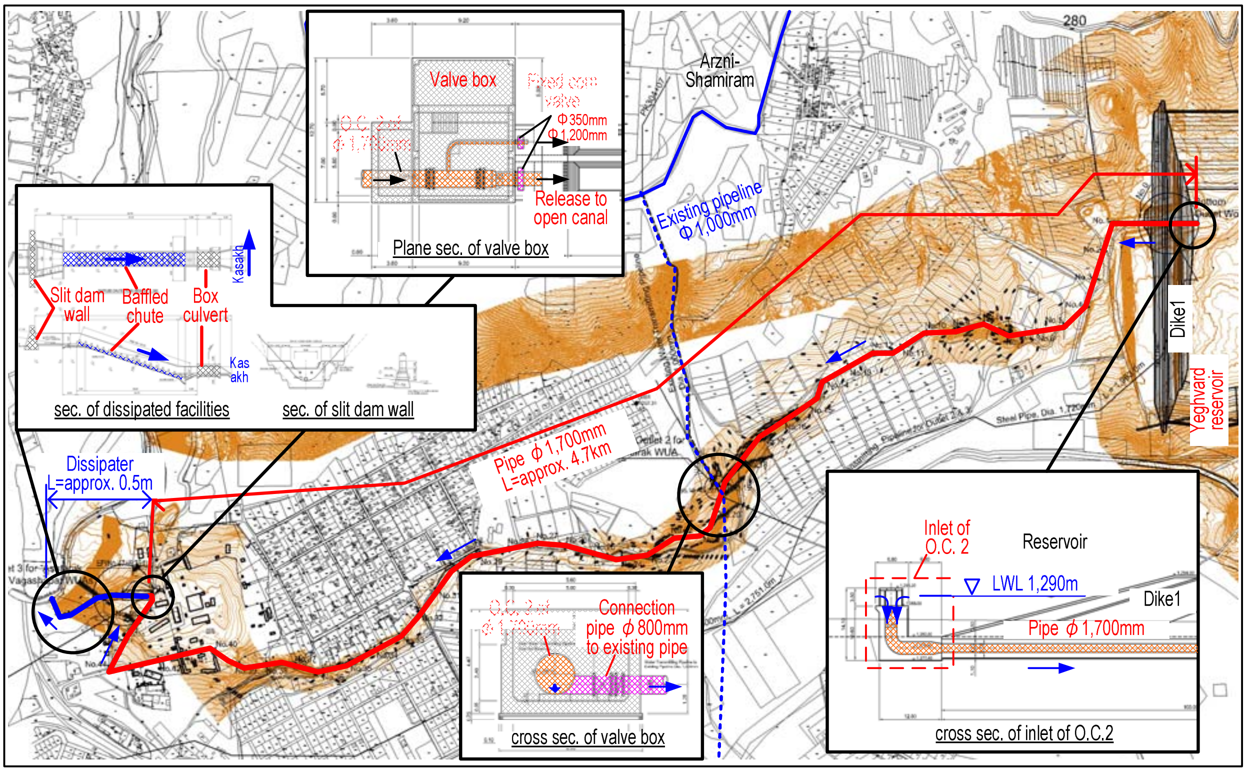


Figure 6-4-4.11 Plan of Outlet Canal 2

6-4-4-2 Improvement plan for Targeted area 2

(1) Outline of rehabilitation plan

The Target area 2 should be improved and rehabilitated as shown in Table 6-4-4.15.

Table 6-4-4.15 Outline of Rehabilitation Plan

<table>
<tr><th>Facility and structure</th><th>Rehabilitation outline</th><th>Responsibility</th></tr>
<tr><td>Arzni-Shmiram canal</td><td>・Section between approx. PK14 and PK17, PK28 and PK32, PK64 and PK69, PK85 and PK93, PK94 and PK96. PK96 and PK97, PK101 and PK105 (L=2.7km)
・Remove concrete panel and line with concrete</td><td rowspan="6">WSA</td></tr>
<tr><td>Lower Hrazdan canal part2,
BP. to PK219</td><td>・Section between PK10 and PK188 (L=17.8km)
・Add the concrete for raising to the sidewall
・Installation of 2 pipes that connect Upper Aknalich canal (φ400mm) at PK10 and Inner Aknalich canal (φ1,000mm) at PK13 with Lower Hrazdan canal at PK188 .</td></tr>
<tr><td>Aknalich PS.</td><td>・Abolished</td></tr>
<tr><td>Metsamor PS</td><td>・Abolished</td></tr>
<tr><td>Ranchaper PS. 1</td><td>・Abolished</td></tr>
<tr><td>Ranchaper PS. 2</td><td>・Abolished</td></tr>
<tr><td>Arzni-Branch canal, BP. to PK120</td><td>・Section between BP and PK23 (L=2.3km)
・Remove the current canal and construct the lining concrete and/or install the precast concrete canal
・Replace 1 gate</td><td>Yeghvard WUA</td></tr>
<tr><td>Arzni-Branch canal, PK120 to EP.</td><td>・Section between PK123 and PK234. (L=12.1km)
・Remove the current canal and construct the lining concrete and/or install the precast concrete canal
・Replace 22 gates, 1 water measurement facility and 2 aqueduct bridges</td><td rowspan="2">Ashtarak WUA</td></tr>
<tr><td>Takahan canal, BP. to PK130</td><td>・Section between PK69 and PK126 (L=5.4km(except pipeline 0.3km)
・Remove the current canal and construct the lining concrete and/or install the precast concrete canal
・Replace 17 gate and 2 aqueduct bridges</td></tr>
<tr><td>Shah-Aru canal, BP. to PK118</td><td>・Section between BP. and PK31 PK62 and PK70, PK82 and PK112 (L=6.9km)
・Remove the current canal and construct the lining concrete and/or install the precast concrete canal
・Replace 16 gates</td><td rowspan="2">Vagharshapat WUA</td></tr>
<tr><td>Inner Aknalich canal</td><td>・No rehabilitation in the Project</td></tr>
<tr><td>Upper Aknalich canal BP to PK104</td><td>・Section between PK6 and PK104 (L=9.8km)
・Install the precast concrete canal in existing canals
・Replace 39 gates and 2 aqueduct bridges</td><td rowspan="2">Khoy WUA</td></tr>
<tr><td>Metsamor canal</td><td>・No rehabilitation in the Project
・Facilities and structures were rehabilitated under the assistance of the World Bank.</td></tr>
</table>

(2) Location of rehabilitation plan

The target improvement and rehabilitation are as shown in Figure 6-4-4.12.

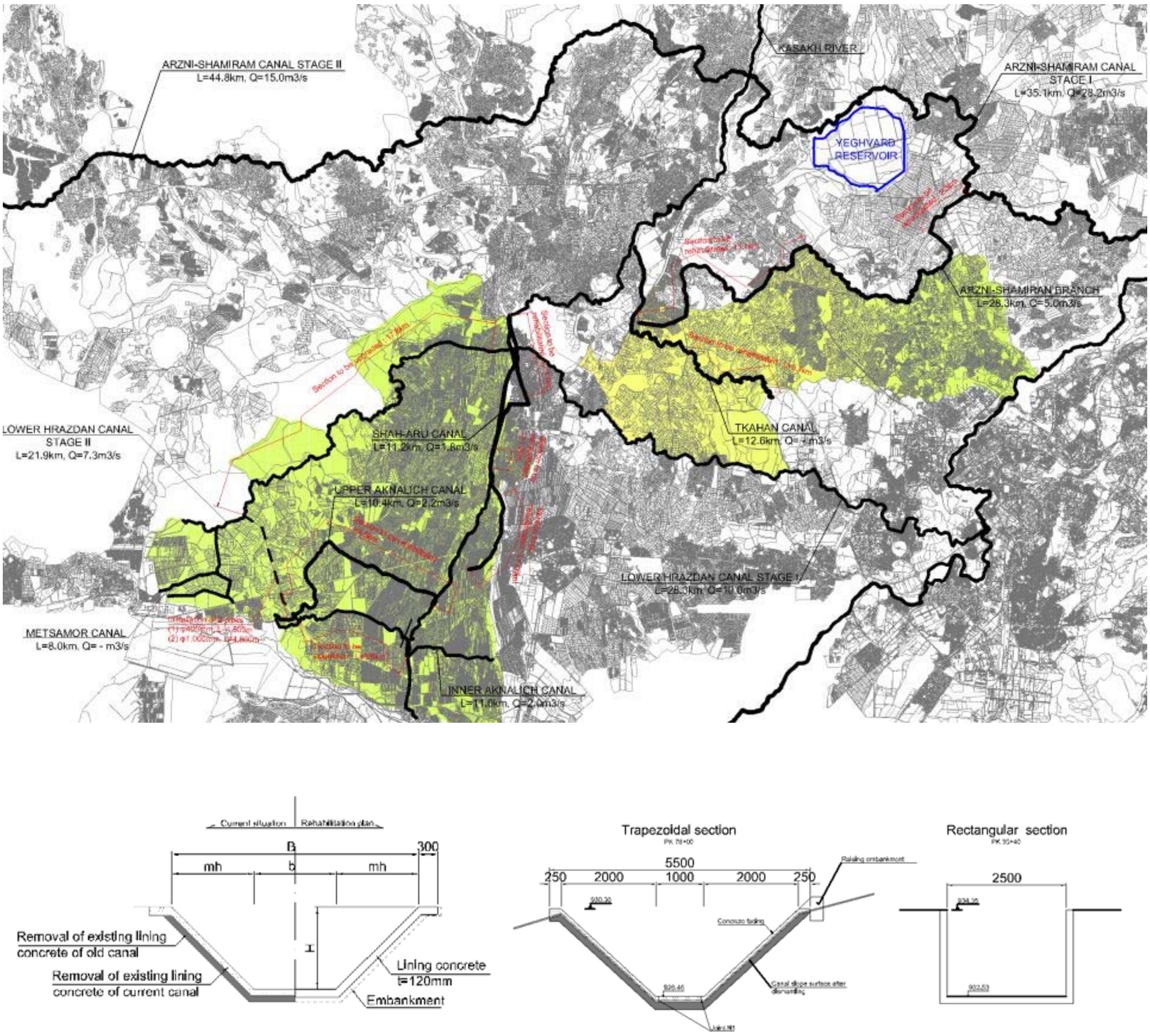


Figure 6-4-4.12 Canal Rehabilitation Plans

6-4-4-3 Plan of abolishment of pump station

As the result of completion of the Project, existing pump facilities are planned to be abolished because of replacement to gravity irrigation by Yeghvard reservoir. Presently, although the area by pump irrigation is mainly Khoy WUA and Vagharshapat WUA, these area are planned to be conveyed the irrigation water through Lower Hrazdan canal part 2 after the Project.

The profitability of the Project is expected to cost cutting of electricity for pumps. The priority of abolishment plan is assumed as followings;

- ✓ First priority : Four of major pump stations as followings and 13 of minor pump stations

  Ranchpar 1 and 2 P.S. for Vagharshapat WUA
  Aknalich P.S. for Khoy WUA and Vagharshapat WUA

Metsamor P.S. for Khoy WUA

- ✓ Second priority : 133 of deep wells scattered in Khoy WUA and Vagharshapat WUA

note) number of pump station and deep wells by survey

The pump irrigation, however, have been settled to present irrigation system so far. Especially, many deep wells could be essential water source as not only agriculture but also supplemental water. Therefore, before the abolishment of pump station, the instruction and explanatory conference should be taken place. In addition, the situation of deep wells should be carefully assessed and the phased abolishment plan could be appropriate to avoid the burden to the farmers.

6-5 Reservoir Plans

6-5-1 Comparative Study of the Reservoir Scale

(1) Facility layout around private orchard area

Northern slope of reservoir has high permeability and protection with anti-infiltration capacity on the slope is needed to reduce leakage volume.

There is an orchard area at the west edge of northern slope and a part of this area is target area to be covered by slope protection. Since this is a private area, some compensation to land owner is required.

On the other hand, it can be considered to extend Dam No.1 along the toe of slope as impervious structure instead of slope protection. In this case, orchard area is free from occupation by any facilities and no compensation is required. However additional cost to construct dam is required.

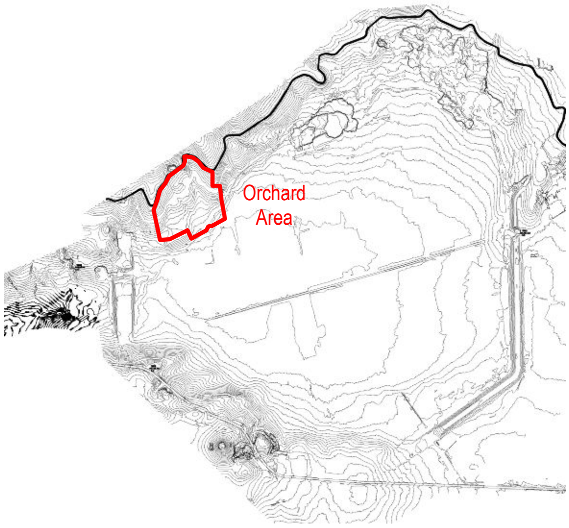
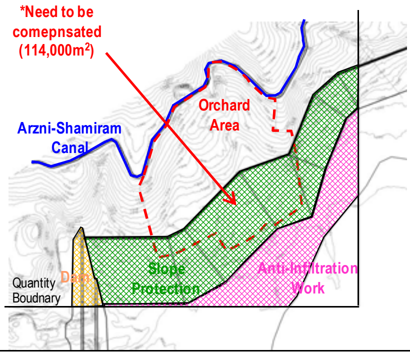
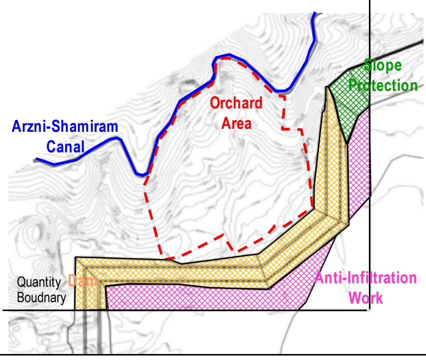


Figure 6-5-1.1 Location of Orchard Area

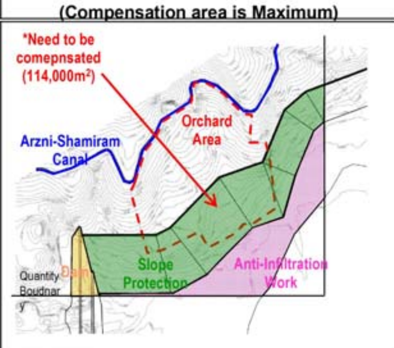
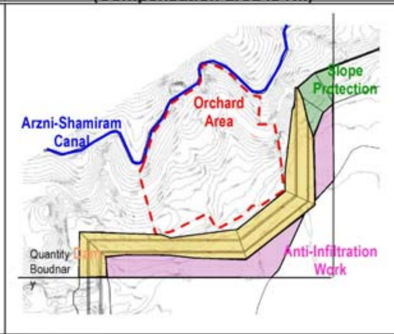
Therefore here conducts an economical comparative study targeting two (2) plans shown in Table 6-5-1.1.

Table 6-5-1.1 Outline of Comparative Plans for Orchard Area

		Plan A	Plan B
Outline			
Leakage Control Structure		Slope protection with anti-infiltration capacity	Dam constructed along the toe of slope
Compensation for Orchard Area		√	
Construction Cost	Slope Protection	√	
	Dam		√

The result of comparative study is shown in Table 6-5-1.2 and Plan A is selected due to economical advantage.

Table 6-5-1.2 Results of Comparison Study for Orchard Area

			Plan A (Compensation area is Maximum)					Plan B (Compensation area is Nil)				
Outline												
Compensation fee			Area/Volume (m^2/m^3)		Unit Cost (USD)		Sub Total (USD)	Area/Volume (m^2/m^3)		Unit Cost (USD)		Sub Total (USD)
Compensation fee		Tree loss	114,000 m^2	x	0.18	=	20,520	0 m^2	x	0.18	=	0
Compensation fee		Land loss	114,000 m^2	x	0.60	=	68,400	0 m^2	x	0.60	=	0
Construction Cost	Slope Protection	Small Dike	10,000 m^3	x	33.14	=	331,400	990 m^3	x	33.14	=	32,809
Construction Cost	Slope Protection	Slope protection	314,000 m^2	x	14.31	=	4,493,340	27,000 m^2	x	14.31	=	386,370
Construction Cost	Anti Infiltration Work		154,000 m^2	x	14.31	=	2,203,740	170,000 m^2	x	14.31	=	2,432,700
Construction Cost	Dam	Core	59,000 m^3	x	4.56	=	269,040	375,000 m^3	x	4.56	=	1,710,000
Construction Cost	Dam	Filter	5,700 m^3	x	11.52	=	65,664	31,000 m^3	x	11.52	=	357,120
Construction Cost	Dam	Surface Protection	7,700 m^3	x	33.14	=	255,178	57,000 m^3	x	33.14	=	1,888,980
Construction Cost	Dam	Sand-and-Gravel	130,000 m^3	x	4.91	=	638,300	919,000 m^3	x	4.91	=	4,512,290
Construction Cost	Dam	Sand-and-Gravel (Dam Cres	1,500 m^3	x	4.91	=	7,365	7,900 m^3	x	4.91	=	38,789
Construction Cost	Dam	Scoria (Dam Crest)	240 m^3	x	4.91	=	1,178	1,300 m^3	x	4.91	=	6,383
Construction Cost	Dam	Counter Weight	7,100 m^3	x	3.83	=	27,193	49,095 m^3	x	3.83	=	188,032
Construction Cost	Dam	Stripping	14,000 m^3	x	3.98	=	55,720	87,000 m^3	x	3.98	=	346,260
Construction Cost	Direct Construction Cost						8,348,118					11,899,733
Construction Cost	InDirect Cost (111% of Direct Cost)						9,266,411					13,208,704
Construction Cost	Sub Total						17,614,529					25,108,437
Total	(USD)						17,703,449					25,108,437
Total	**(Million USD)**						**17.7**					**25.1**

(2) Facility layout to reduce total construction cost

A part of reservoir bottom, north slope and south slope has high permeability and anti-infiltration work is required on them. Total area of those are very huge and the cost of anti-infiltration works account high ratio of total construction cost.

Therefore in addition to examine economical anti-infiltration work structure, facility layout to reduce the area of anti-infiltration area shall be examined as well.

The most simple structure is to cover all the target area by anti-infiltration work.

On the other hand, it can be consider to construct dams along the toe of slope. In this case, anti-infiltration work on slope and a part of reservoir bottom is not required. However another construction cost, dam construction cost, is required. Additionally, since reservoir area becomes narrow, FWL (Full Water Level) shall be raised up to keep necessary capacity and the height of dam becomes higher.

Therefore here conducts an economical comparative study targeting two (2) plans shown in Table 6-5-1.3.

Table 6-5-1.3 Outline of Comparative Plans to Minimize Anti-Infiltration Area

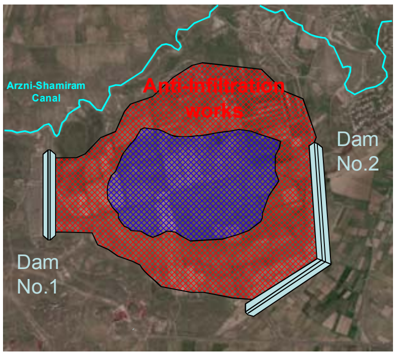
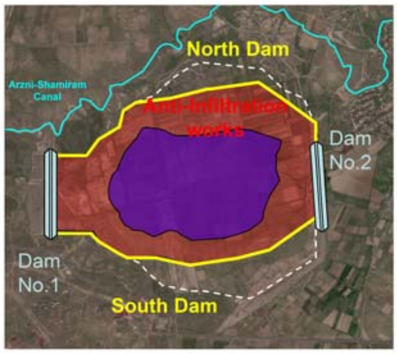
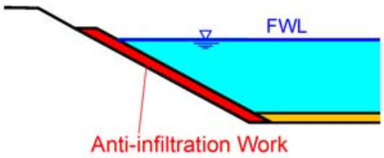
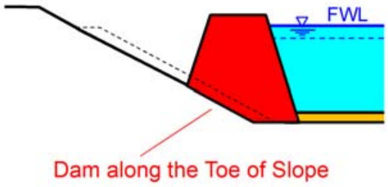
		Plan A	Plan B
Outline	Plan		
	Leakage Control Structure for Slope Area		
FWL		Low	High
Dam height		Low	High
Area of Anti-infitration works	North Slope	Huge	Nil
	South Slope	Huge	Nil
	Reservoir Bottom	Huge	Less than Plan A
Cost to construct dams along the toe of slope		Nil	High
Others		Material for dam body is collected from the area within reservoir area	Material for dam body is collected from area within and out of reservoir area. *Amount of material within reservoir area is not enough

Figure 6-5-1.2 and 6-5-1.3 illustrate plan and typical cross section of each plan and result of comparative study is shown in Table 6-5-1.4.

Finally Plan A is selected due to economical advantage.

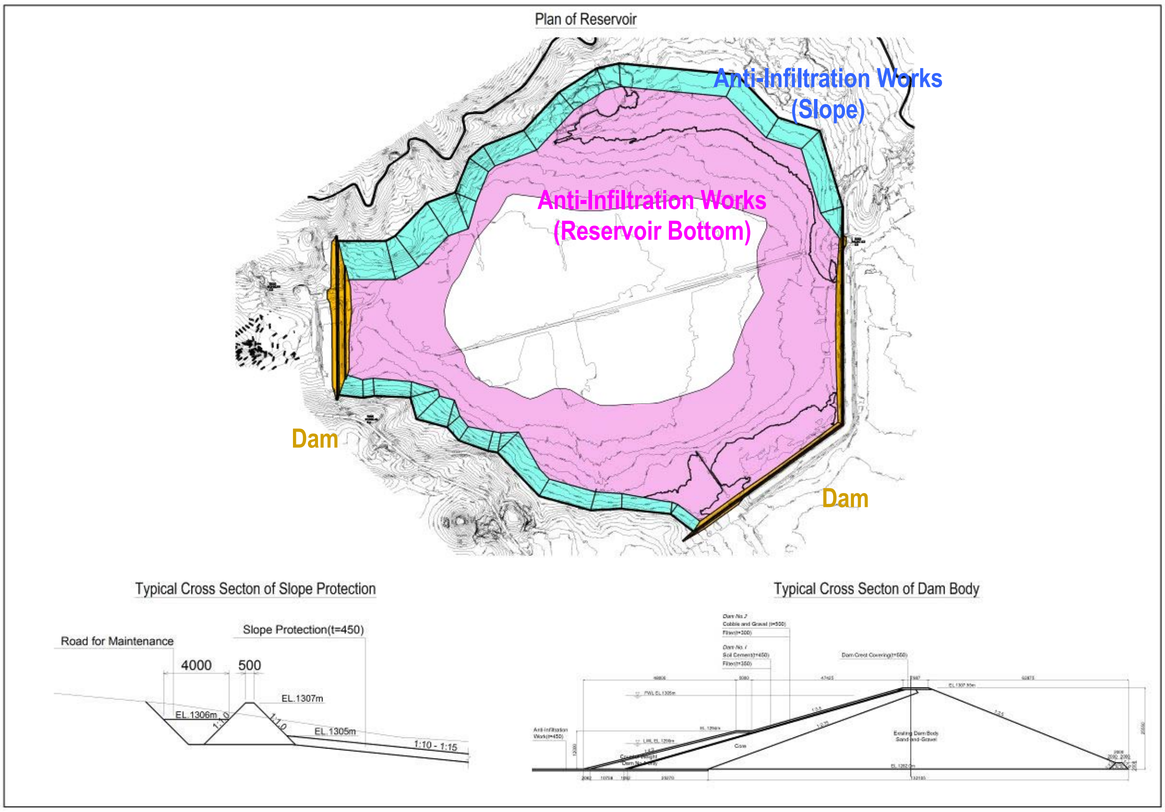


Figure 6-5-1.2 Plan and Typical Cross Section (Plan-A)

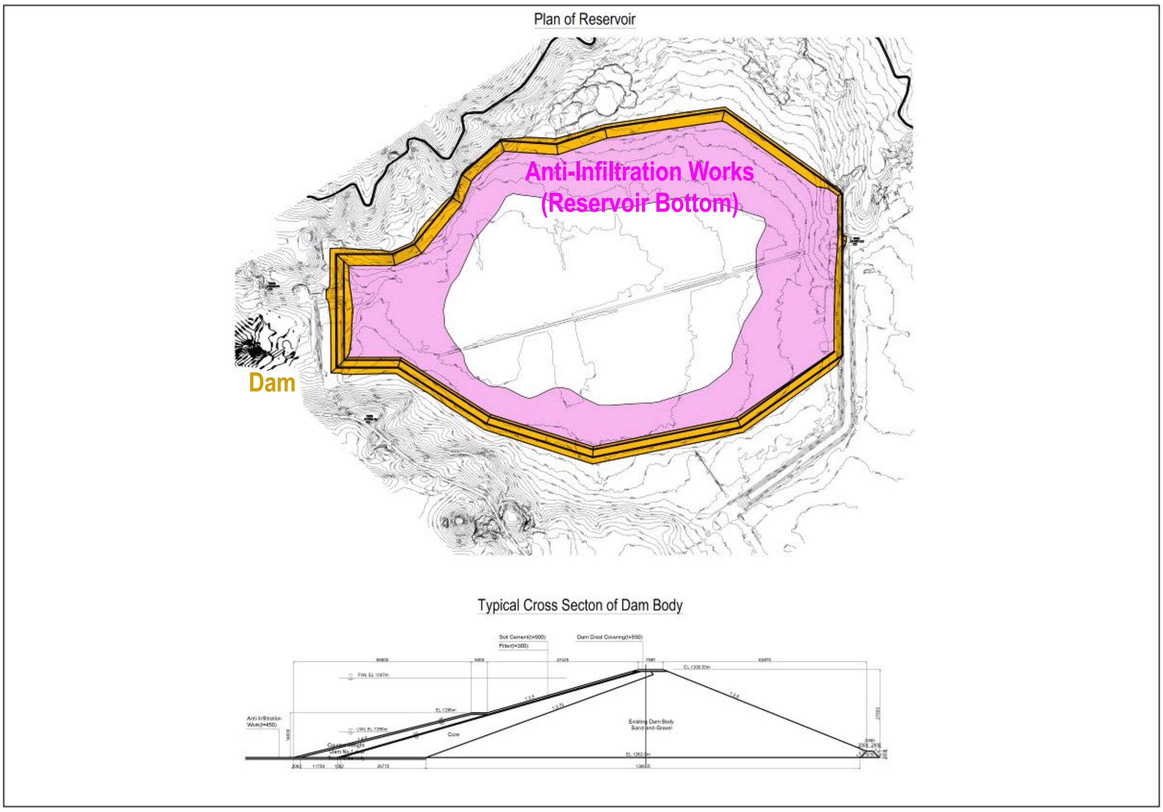
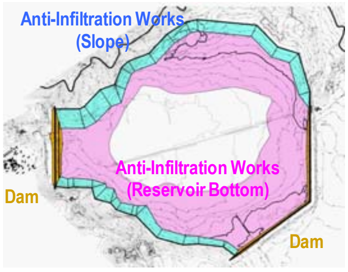
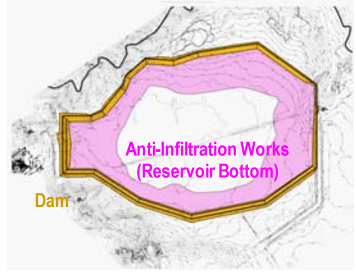


Figure 6-5-1.3 Plan and Typical Cross Section (Plan-B)

Table 6-5-1.4 Results of Comparison Study to Minimize Anti-Infiltration Area

	Plan A	Plan B
	900ha	600ha
Outline		

					Plan A Quantity			Plan A Unit Cost (USD)		Plan A Amount (USD)	Plan B Quantity			Plan B Unit Cost (USD)		Plan B Amount (USD)
Reservoir Properties	Reservoir Capacity									94 MCM						Same as on the left
Reservoir Properties	LWL									EL. 1290m						Same as on the left
Reservoir Properties	FWL									EL. 1305m						EL. 1307m
Reservoir Properties	Dam Height									25.55m						27.55m
Reservoir Properties	Reservoir Area									7.96km^2						5.42km^2
Direct Construction Cost	Items				Quantity			Unit Cost (USD)		Amount (USD)	Quantity			Unit Cost (USD)		Amount (USD)
Direct Construction Cost	Dam	No.1	Filling	Core	281,000	m^3	×	4.56	=	1,281,360	295,000	m^3	×	4.56	=	1,345,200
	Dam	No.1	Filling	Filter	25,000	m^3	×	11.52	=	288,000	24,000	m^3	×	12.12	=	290,880
	Dam	No.1	Filling	Slope protection (Soil Cement)	36,000	m^3	×	33.14	=	1,193,040	35,000	m^3	×	33.86	=	1,185,100
	Dam	No.1	Filling	Sand-and-Gravel	528,000	m^3	×	4.91	=	2,592,480	596,000	m^3	×	5.56	=	3,313,760
	Dam	No.1	Filling	Sand-and-Gravel (Dam Crest)	5,200	m^3	×	4.91	=	25,532	3,700	m^3	×	5.56	=	20,572
	Dam	No.1	Filling	Scoria (Dam Crest)	850	m^3	×	4.91	=	4,174	620	m^3	×	5.56	=	3,447
	Dam	No.1	Filling	Counter Weight	47,000	m^3	×	3.83	=	180,010	59,000	m^3	×	3.83	=	225,970
	Dam	No.1	Stripping		47,000	m^3	×	3.98	=	187,060	43,000	m^3	×	3.98	=	171,140
	Dam	No.1	Sub-Total							5,751,656						6,556,069
	Dam	No.2	Filling	Core	247,000	m^3	×	4.56	=	1,126,320	153,000	m^3	×	4.56	=	697,680
	Dam	No.2	Filling	Filter	26,000	m^3	×	11.52	=	299,520	14,000	m^3	×	12.12	=	169,680
	Dam	No.2	Filling	Slope protection (Cobble and Gravel)	43,000	m^3	×	4.74	=	203,820	24,000	m^3	×	5.39	=	129,360
	Dam	No.2	Filling	Sand-and-Gravel	62,000	m^3	×	4.91	=	304,420	50,000	m^3	×	5.56	=	278,000
	Dam	No.2	Filling	Sand-and-Gravel (Dam Crest)	14,000	m^3	×	4.91	=	68,740	5,000	m^3	×	5.56	=	27,800
	Dam	No.2	Filling	Scoria (Dam Crest)	2,000	m^3	×	4.91	=	9,820	830	m^3	×	5.56	=	4,615
	Dam	No.2	Stripping		40,000	m^3	×	3.98	=	159,200	23,000	m^3	×	3.98	=	91,540
	Dam	No.2	Sub-Total							2,171,840						1,398,675
	Dam	North	Filling	Core							561,000	m^3	×	4.56	=	2,558,160
	Dam	North	Filling	Filter							54,000	m^3	×	12.12	=	654,480
	Dam	North	Filling	Slope protection (Cobble and Gravel)							90,000	m^3	×	5.39	=	485,100
	Dam	North	Filling	Sand-and-Gravel							1,166,000	m^3	×	5.56	=	6,482,960
	Dam	North	Filling	Sand-and-Gravel (Dam Crest)							18,000	m^3	×	5.56	=	100,080
	Dam	North	Filling	Scoria (Dam Crest)							3,000	m^3	×	5.56	=	16,680
	Dam	North	Stripping								107,000	m^3	×	3.98	=	425,860
	Dam	North	Sub-Total													10,723,320
	Dam	South	Filling	Core							1,009,000	m^3	×	4.56	=	4,601,040
	Dam	South	Filling	Filter							91,000	m^3	×	12.12	=	1,102,920
	Dam	South	Filling	Slope protection (Soil Cement)							130,000	m^3	×	33.86	=	4,401,800
	Dam	South	Filling	Sand-and-Gravel							2,758,000	m^3	×	5.56	=	15,334,480
	Dam	South	Filling	Sand-and-Gravel (Dam Crest)							17,000	m^3	×	5.56	=	94,520
	Dam	South	Filling	Scoria (Dam Crest)							2,800	m^3	×	5.56	=	15,568
	Dam	South	Filling	Counter Weight							129,000	m^3	×	3.83	=	494,070
	Dam	South	Stripping								190,000	m^3	×	3.98	=	756,200
	Dam	South	Sub-Total													26,800,598
	Anti-infiltratio works	North Slope	Small dike (Soil Cement)		37,000	m^3	×	33.14	=	1,226,180						
	Anti-infiltratio works	North Slope	Slope Protection with anti-infiltration capacity		807,000	m^2	×	14.31	=	11,548,170						
	Anti-infiltratio works	North Slope	Sub-Total							12,774,350						
	Anti-infiltratio works	South Slope	Small dike (Soil Cement)		22,000	m^3	×	33.14	=	729,080						
	Anti-infiltratio works	South Slope	Slope Protection with anti-infiltration capacity		354,000	m^2	×	14.31	=	5,065,740						
	Anti-infiltratio works	South Slope	Sub-Total							5,794,820						
	Anti-infiltratio works	Reservoir Bottom			4,282,000	m^2	×	14.31	=	61,275,420	3,101,000	m^2	×	14.31	=	44,375,310
	Total		(USD)							87,768,086						89,853,972
	Total		(Million USD)							87.8						89.9

6-5-2 Estimation of Leakage Rate from Reservoir

The Yeghvard reservoir is a wide-spread flat-basin type reservoir constructed on a foundation which mainly consists of volcanic rocks and sediments. Since the foundation is mostly pervious and the groundwater level is very low, it is obvious that an artificial anti-infiltration layer must be placed on the basin of the reservoir to reduce water leakage. Therefore, in order to grasp the efficiency of the layer, the leakage rate was estimated for alternative cases of reservoir layout and covering extent of the anti-infiltration layer.

(1) Method

Two (2) methods are applied, namely; the "2-D Simple Method" and the "3-D FEM Method". The calculation for all alternative cases was carried out with the 2-D method. The 3-D method was applied only for the main cases to infer the three-dimensional flow condition.

(a) 2-D simple method

Considering the hydrogeological conditions and the large extent of reservoir, it is hydraulically apparent that the reservoir water mainly infiltrates and flows down almost vertically to the groundwater body located deep. Therefore, there is no significant error if we consider only the vertical flow for the leakage rate estimation. The "2-D Simple Method" is one of such methods based on the Darcy's law which is basically the same as used in the detail design, 1985 in Soviet era.

First, the reservoir area was divided into triangles about 60m wide and tall as shown in Figure 6-5-2.1 **(**Such a set of area-covering triangles is called a "TIN - Triangulated Irregular Network"). Then the vertical infiltration rate is calculated at each triangle with the average thicknesses of geologic layers, their representative coefficients of vertical permeability, reservoir water depth and area of triangle. The total infiltration (leakage) from the reservoir is calculated by summing the rates for all triangles.

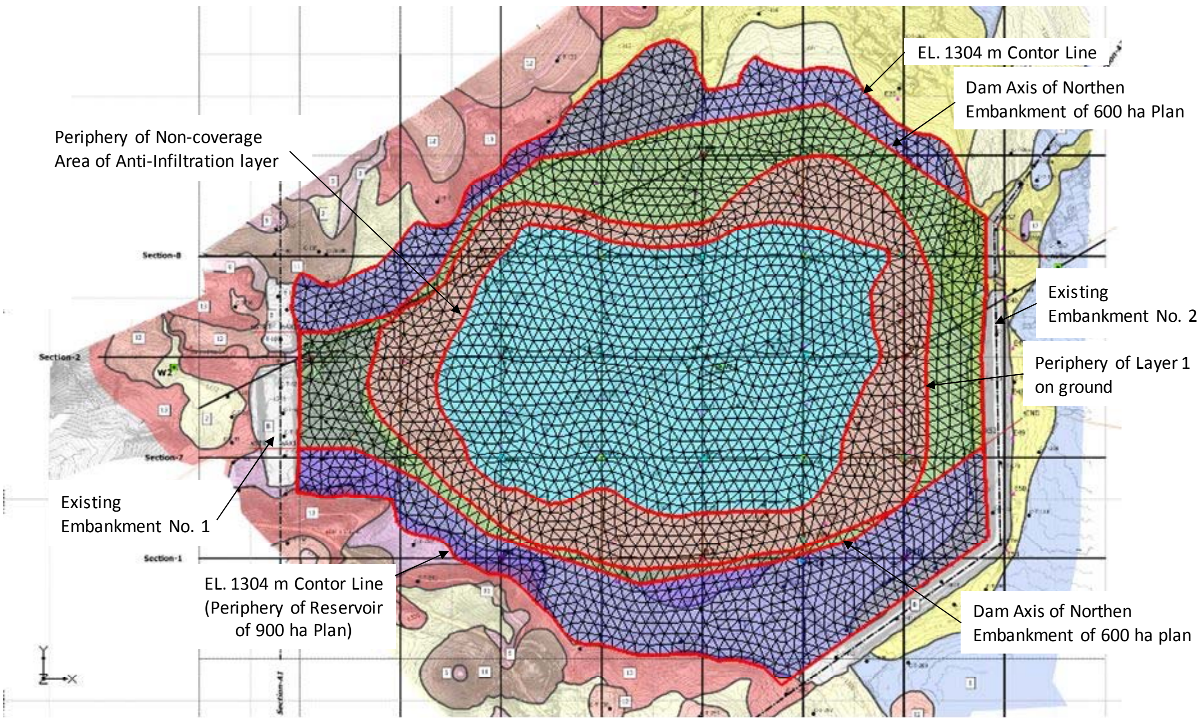


Figure 6-5-2.1 TIN for Calculation of Infiltration Rate from Reservoir

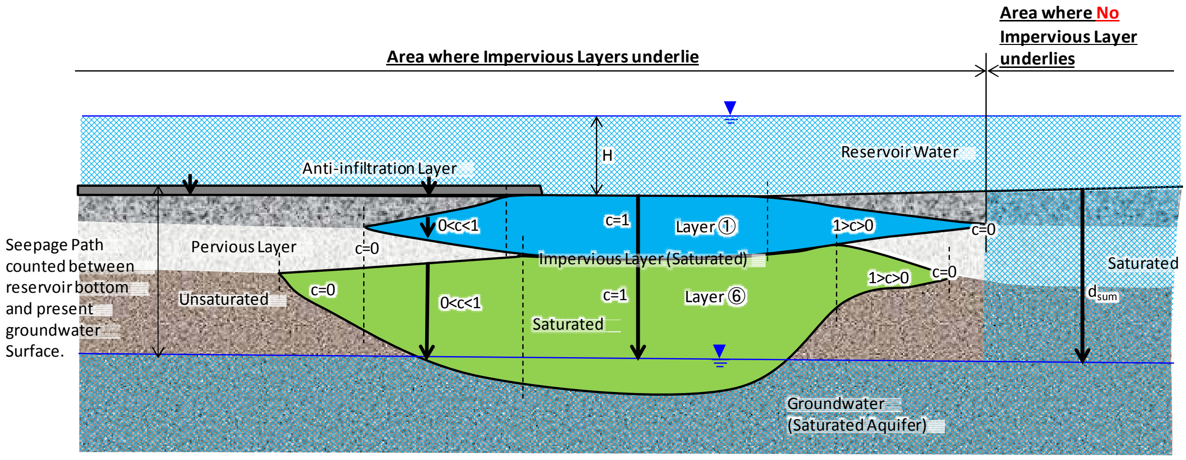


Figure 6-5-2.2 Schematic Figure to Explain Assumption of Infiltration-Rate Calculation Formula

The calculation formula is as follows:

$$Q_{total} = \sum_{i=1}^{n} Q_i$$
$$Q_i = q_i A_i$$

where Q_{total}: Total infiltration rate from reservoir
Q_i: Infiltration rate in an element
A_i: Area of an element
q_i: Specific infiltration rate at an element (rate in a unit area)
n: Number of elements

$$q_i = k_{ave} \frac{H + d_{sum}}{d_{sum}}$$

where k_{ave}: Average coefficient of vertical permeability of layers under an element (Harmonic mean weighted with layer thickness)
d_{sum}: Total average thickness of layers under an element
H: Average water depth on an element

$$k_{ave} = \frac{\sum c_i d_i}{\sum \frac{c_i d_i}{k_i}} \qquad d_{sum} = \sum c_i d_i$$

where k_i: Coefficient of vertical permeability of layer in saturation
d_i: Thickness of layer
c_i: Reduction factor of thickness of layer (see the Figure 6-5-2.2)

The seepage path is counted between the reservoir bottom and the present groundwater surface. The target layers for the calculation were selected as follows:

Area where impervious layers underlie

- Seepage path through only impervious layers (the Layer 1, Layer 6 and the anti-filtration layer) were considered.
- Seepage path through pervious layers were ignored, assuming that lateral or unsaturated flow

occurs in them.

- In case, a pervious layer overlies on an impervious layer, a reduction factor ranging 0 to 1 as shown in Figure 6-5-2.2 is applied to the thickness of the impervious layer assuming that the water partially flows out laterally.

Area where no impervious layer underlies

- Seepage path through all layers considered, assuming that saturated flow occurs.

In an area where an impervious layer is thin, the infiltration rate considering only the impervious layer might exceed that considering all layers. In that case, the latter value is adopted, because it means saturated flow occurs through the layers.

(b) 3-D FEM method

The 3-D FEM Method is the three-dimensional saturated and unsaturated seepage flow analysis with the finite element method. It formulates the groundwater flow with the calculus formula based on the Darcy's Law for the saturated zone and the so-called Richards's formula for the unsaturated zone. For the steady state, the formula is written as follows:

$$\nabla \cdot (\mathrm{K} \cdot \nabla \mathrm{h}) = 0$$

where: h = total head (elevation head plus pressure head)

K = hydraulic conductivity (coefficient of permeability)

$$\mathrm{K} = \mathrm{K}_{sat} \cdot \mathrm{k}_{r}$$

K_{sat} = hydraulic conductivity in saturation

k_r = relative permeability (=1 in saturated zone; >0 and <1 in unsaturated zone)

In the unsaturated zone, the Darcy's Law is also applied, but the coefficient of permeability is multiplied by a relative permeability ranging greater than 0 to 1 which relates to the moisture content or suction head in a layer. The program code used for the calculation is the "DTRANS-3D" which is developed by Prof. M. Nishigaki of Okayama University, Mitsubishi Material Co. Ltd. and Dia Consultant Co. Ltd.

Figure 6-5-2.3 shows the FEM mesh for the calculation. It consists of the prism elements which are extruded from the same TIN in the reservoir area as used for the 2-D simple method.

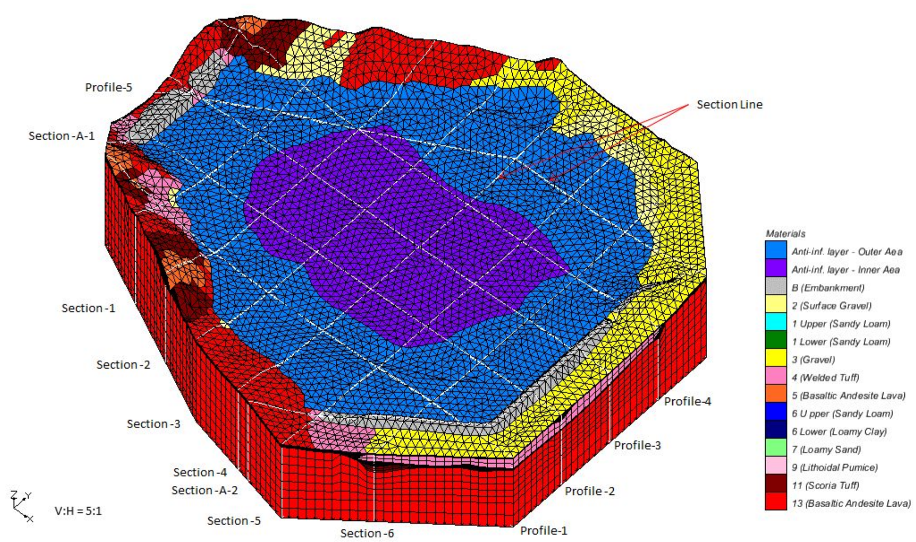


Figure 6-5-2.3 3-D Mesh used for Infiltration-Rate Calculation with 3-D FEM Method

(2) Basic conditions

Ground elevation

The ground elevation of the TIN is interpolated with the surveyed topographic map contours. .

Boundary elevation of geologic layers

The boundary elevation of the geology layers are interpolated from the geological map and sections made with the results of the present and past geological investigations. Figure 6-5-2.4 shows a 3-D geology model developed with the boundary elevation of layers used in the leakage calculation

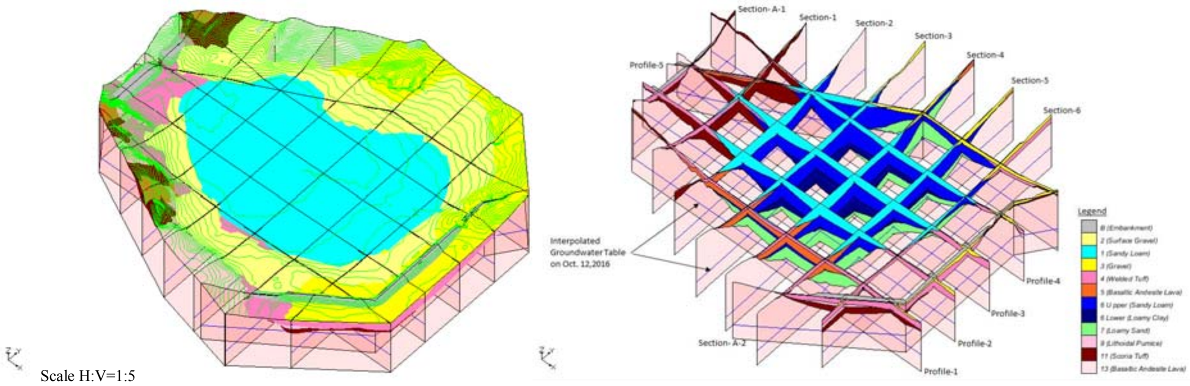


Figure 6-5-2.4 3-D Geology Model Developed with Boundary Elevations Used in Leakage Calculation

Permeability coefficient of geologic layers

The geometric mean of the values obtained by in-situ permeability tests is used for each layer as shown in Table 6-5-2.1. The used vertical permeability coefficient of the layer 1 is 4.3×10^{-4} cm/s for the upper 10m and 8.3×10^{-6} cm/s for the lower. That of the layer 6 upper is 7.5×10^{-6}cm/s, which is also used for the lower for the safety side.

Table 6-5-2.1 Average Coefficient of Permeability of Geologic Layers

Impervious Layers

Layer No.		Main Lithology	Test Place	Test by JICA, 2015						Test in the D/D, 1985					
				No. of Data	Vertical		No. of Data	Horizontal		No. of Data	Vertical		No. of Data	Horizontal	
D/D 1985	JICA 2015				Arithmetic Mean	Geometric Mean		Arithmetic Mean	Geometric Mean		Arithmetic Mean	Geometric Mean		Arithmetic Mean	Geometric Mean
①	①	Sandy Loam/Loam	Testpit	20	5.1E-04	4.3E-04	20	1.6E-03	1.2E-03	28	3.1E-04	2.1E-04	-	-	-
			Borehole	46	1.6E-05	8.3E-06	41	1.0E-04	4.9E-05	-	-	-	6	1.7E-05	9.7E-06
			All	-	-	-	-	-	-	-	-	-	34	2.0E-04	7.1E-05
⑥	⑥Upper	Loamy Sand/ Loam	Borehole	57	4.5E-05	7.5E-06	50	2.9E-04	3.2E-05	-	-	-	6	1.2E-05	1.1E-05
	⑥Lower	Loamy Clay	Borehole	6	3.1E-06	1.3E-06	5	1.3E-05	2.8E-06	-	-	-	-	-	-

Pervious Layer

Layer No.		Main Lithology	Test Place	Test by JICA, 2015						Test in the D/D, 1985					
				No. of Data	Vertical		No. of Data	Horizontal		No. of Data	Vertical		No. of Data	Horizontal	
D/D 1985	JICA 2015				Arithmetic Mean	Geometric Mean		Arithmetic Mean	Geometric Mean		Arithmetic Mean	Geometric Mean		Arithmetic Mean	Geometric Mean
②a	②	Gravel	Borehole	6	4.2E-05	1.7E-05	7	9.2E-04	6.9E-04	-	-	-	4	1.6E-03	1.3E-03
③	③	Gravel	Borehole							-	-	-	19	5.0E-03	1.5E-03
④	④	Welded Tuff	Borehole	13	1.2E-04	1.7E-05	12	5.4E-04	2.3E-04	-	-	-	60	4.7E-03	8.9E-04
⑤	⑤	Lava	Borehole	5	1.1E-05	4.2E-06	4	2.7E-04	2.2E-04	-	-	-	14	8.0E-03	5.4E-04
⑦	⑦	Sand, Sandy Loam	Borehole	12	2.6E-04	3.1E-05	7	5.4E-05	4.6E-05	-	-	-	23	3.1E-03	1.4E-03
⑧		Loamy Sand/Loam	Borehole							-	-	-	5	3.2E-04	-
⑨	⑨	Lithoidal Pumice	Borehole	-	-	-	-	-	-	-	-	-	5	4.4E-04	2.1E-04
⑩	⑩	Welded Tuff	Borehole	-	-	-	-	-	-	-	-	-	5	4.7E-03	-
⑪	⑪	Scoria Tuff	Borehole	-	-	-	-	-	-	-	-	-	64	1.6E-02	1.4E-03
⑬	⑬	Lava	Borehole	21	2.0E-05	6.9E-06	15	3.2E-04	2.0E-04	-	-	-	53	9.8E-03	3.9E-03
⑮		Lava	Borehole							-	-	-	21	2.8E-03	2.0E-03

注） Average is weighted with the test interval.
Gray-colored Layers are volcanic effusives. Generally jointed or pourous.
Yellowe-painted values are used in the present leakage calculation.
Green-colored values are used as the horizaontal permeability in 3-D FEM method..
Red-colored values were used in the leakage calculation in D/D, 1985.

Groundwater level

The groundwater level was interpolated from the observed data at the monitoring wells on Oct. 12, 2016. This was used as the lower limit of seepage path for the calculation by the 2-D simple method and as the fixed head boundary on the saturated part of side face for the 3-D FEM simulation.

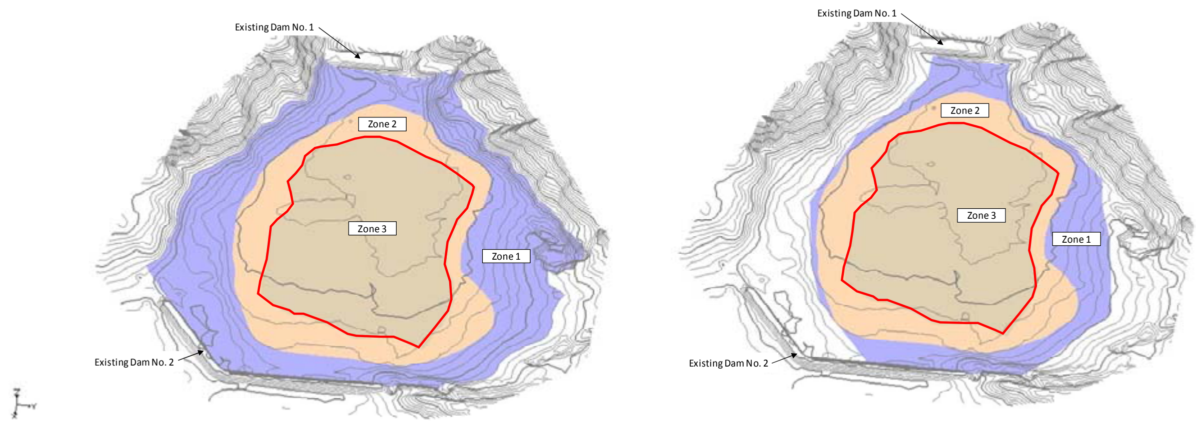
(3) Estimation cases

Figure 6-5-2.5 shows the areal extent of the estimation cases. For the reservoir layout, 900 ha and 600 ha plans were assumed. For the covering extent of the anti-infiltration layer, the whole and the partial coverage cases are assumed. The partial case doesn't cover the central part of the reservoir shown in Figure 6-5-2.6, which is defined with the following conditions:

- Thickness of the layer 1 is greater than 15 m.
- Combined thickness of the layer 1 and 6 is greater than 30 m.
- Distance from the boundary of the layer 1 on the ground is greater than 100 m.

The imperviousness of the anti-infiltration layer is k=1.0 x 10^{-7} cm/s and t=0.2 m, or equivalent. As the full water level of the reservoir corresponding to the capacity 94 MCM, EL. 1304.51 is used for the 900 ha plan and EL. 1306.93 for the 600 ha plan which were calculated with the ground TIN.

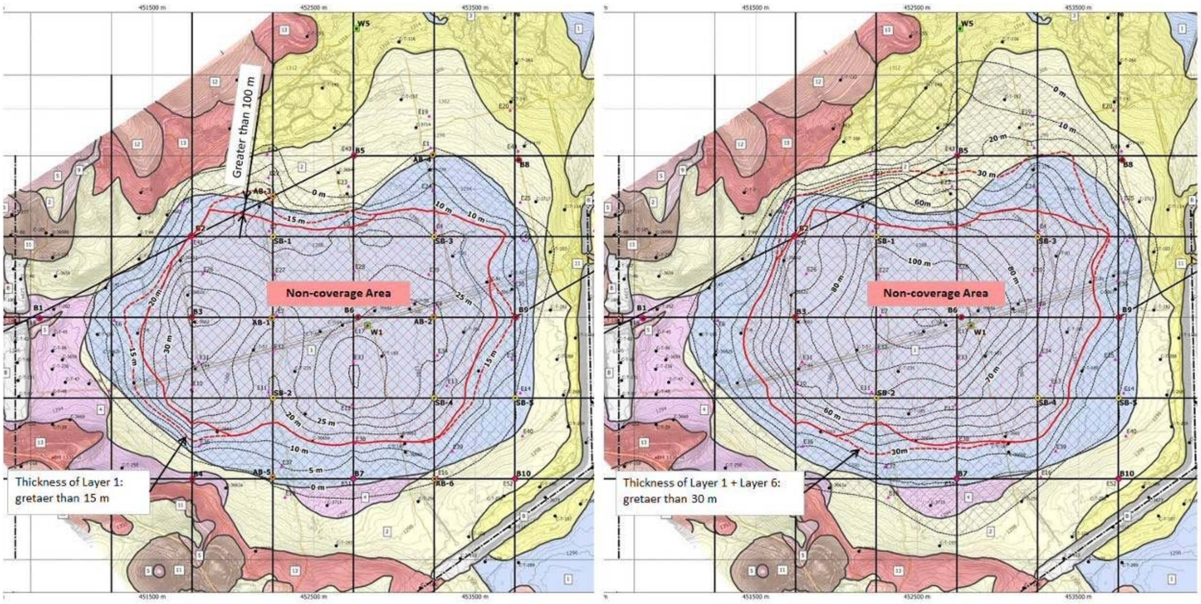
Note) The average permeability coefficient of the layer 1 and 6 is around k=1.0x10^{-5} or less. A layer with k=1.0 x 10^{-5} cm/s and t=20 m has the same imperviousness as the assumed anti-infiltration layer, if the hydraulic head of the layer base is the same. Because the upper about 10m of the layer 1 would have a larger permeability, at least 30 m of thickness would be required to obtain the ability.



900 ha Plan　　600 ha Plan

Note) The Layer 1 (sandy loam) exposes on the ground in the zone 2 and 3. The partial coverage case of anti-infiltration layer doesn't cover the zone 3 (263 ha).

Figure 6-5-2.5 Areal Extent of Estimation Cases



Thickness Contor Map of Layer 1 | Thickness Contor Map of Layer 1 and Layer 6 Combined

Figure 6-5-2.6 Setting of Non-coverage Area of Anti-infiltration Layer

(4) Estimated leakage rate

Table 6-5-2.2 shows the estimated leakage rate from the reservoir for the alternative cases. The leakage rate is estimated around 43,000 to 53,000 m^3/day at the full water level and average 29,000 to 35,000 m^3/day in the irrigation period of the standard year. The value itself is a little large but probably does not affect the reservoir function significantly, because the ratio to the full reservoir capacity – 94 MCM is near or smaller than 0.05%/day which is the Japanese guideline for reservoir construction.

The reservoir water loss is larger in 900 ha plan than 600 ha plan, but the difference is small. Also the difference is not so large between the whole and partial coverage cases of anti-infiltration layer. Therefore the central part of the reservoir, where probably-impervious layers underlie, may not be covered with the anti-infiltration layer considering the cost efficiency.

Table 6-5-2.2 Estimated Leakage Rate from the Reservoir

Reservoir Layout Plan	Anti-infiltration Layer Coverage	Infiltration Rate at 94MCM			Average Infiltraion rate in Irrigation Period of Standard Year		
		Amount (m^3/day)	Ratio	Ratio to 94 MCM (%/day)	Amount (m^3/day)	Ratio	Ratio to 94 MCM (%/day)
900 ha	Whole	45,900	100%	0.049	29,599	100%	0.031
	Partial	52,196	114%	0.056	34,614	117%	0.037
600 ha	Whole	43,190	94%	0.046	28,809	97%	0.031
	Partial	49,712	108%	0.053	33,908	115%	0.036

(5) Flow pattern and movement of infiltrated water

Figure 6-5-2.7 shows the flow pattern and movement of the infiltrated water from the reservoir on the north-south and the east-west sections. The water flows down almost vertically and, after reaching the groundwater body, flows laterally.

Whatever the central part of the reservoir is covered with the anti-infiltration layer or not, the flow pattern in the foundation doesn't change much as understood from the figures. This means that the layer 1 and 6 will work well as natural impervious layers.

The infiltrated water is not useless, but would be a good groundwater recharge for the downstream area.

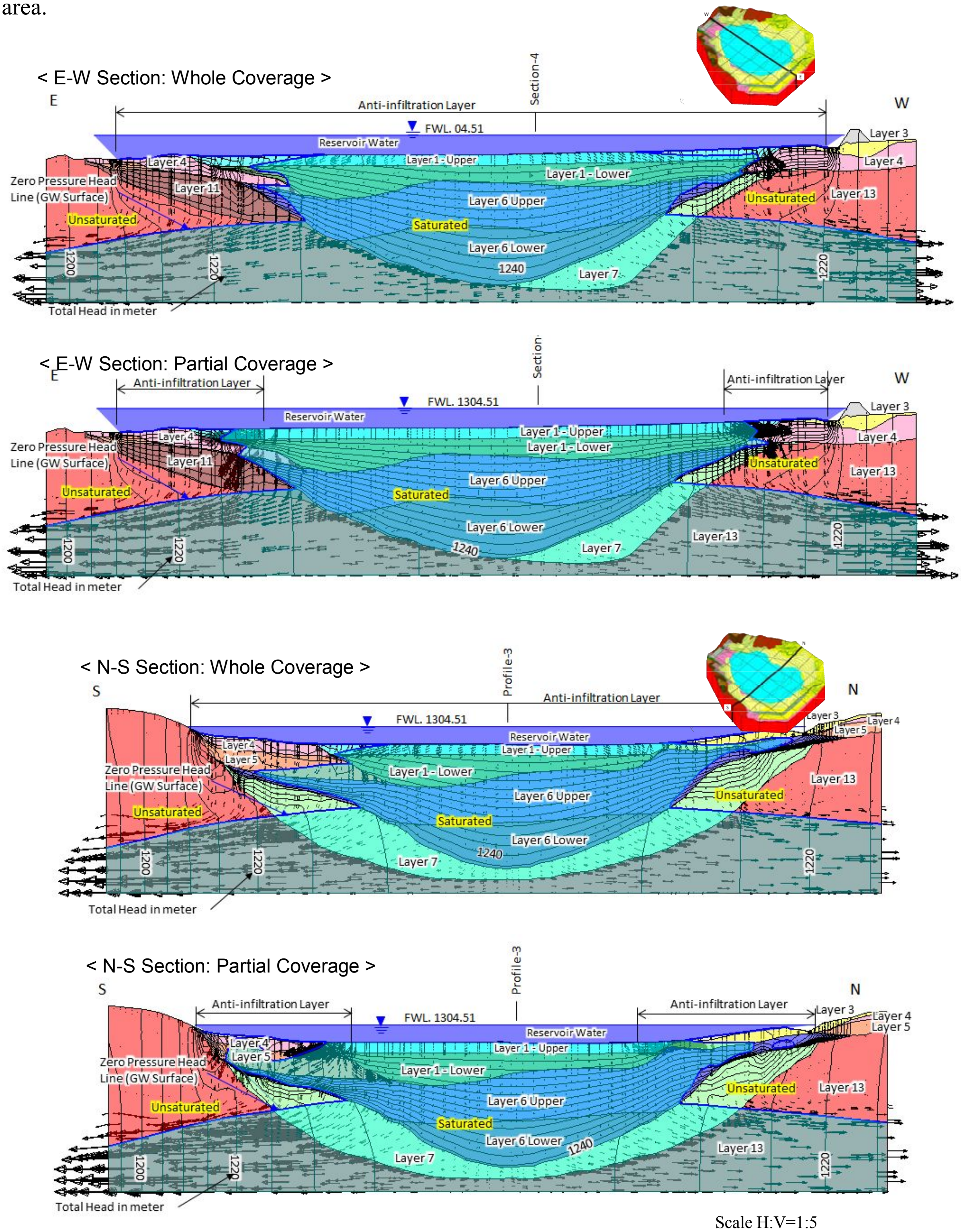


Figure 6-5-2.7 Flow Pattern of Infiltrated Reservoir Water

6-5-3 Outline of the Reservoir Plan

(1) Natural and structural conditions

(a) Meteorological conditions

1) Temperature

- 30 years (1983 - 2012) average of monthly mean temperature = - 4.8 - 23 ℃
- Months with minus (-) monthly mean temperature are December, January and February

2) Precipitation

- Average annual precipitation for 30 years (1983 - 2012) = 445 mm / year
- 30 years (1983 - 2012) average of monthly rainfall are 13mm (August) - 65mm (April)

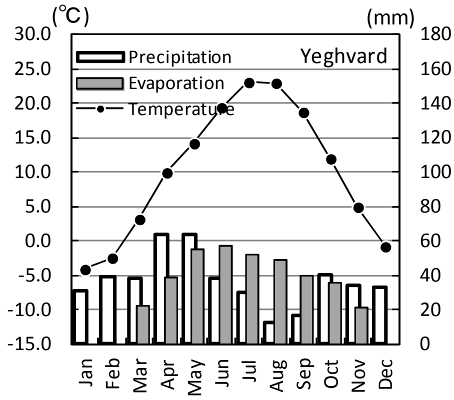


Figure 6-5-3.1 Temperature, Precipitation and Evaporation (1983~2012)

3) Wind

- Maximum mean wind velocity for ten minutes is 18 m/sec
- Predominant wind direction is north to south - north-east to south-west

It is impressive that strong wind keep blowing ceaselessly from north or north-east and the temperature becomes high to be 35℃ or so in summer day-time.

(b) Topographical condition

The reservoir area expanding 3 km long from north to south and 3 km wide from east to west is composed of the wide central plane and gentle slopes at both northern and southern side with the inclination of 1 to 100 or so in average. The reservoir area is pegged out by the existing dam bodies at both eastern and western sides.

(c) Geological condition

The north slopes are composed of Surface Gravel layer, Moraine Deposit, Basaltic Andesite lava stratum and Pyroclastic Flow deposit geologically, all of which are pervious fundamentally. The south slopes have a tri-laminar structure, the first of which is the thin Surface Gravel layer, the second Welded Tuff layer and the third Basaltic Andesite lava layer. The Welded Tuff layer tends to be impervious in case of the layer having no cracks. The basement of the central plane is composed of sediments of loamy soils with thickness more than 120 m approximately at its center. It is revealed by the investigations done in this time that these sediments layers have relatively low permeability coefficient and clear anisotropy between horizontal and vertical permeability.

(d) Structural condition

The expected reservoir capacity is about 94,000,000 m^3 on 9,400,000m^2 of the approximate reservoir area. It would be able to say that the reservoir is a shallow pond with tremendous expansion of 9,400,000 m^2 of water surface.

(2) Topic items to be considered in the reservoir planning

(a) Slope protection against wave actions

Strong wind and the long blow-over distance bring high waves to reservoir slopes so that they shall be protected against wave actions by the protection work.

(b) Protection against the freezing-thawing effect

Low temperature less than 0 ℃ in average from December to February shall bring the cycles of freezing and thawing that would loosen the compacted soil layer to be weak in strength and be more pervious in seepage. To avoid such circumstances brought to the compacted soil layers, suitable protection works shall be provided with on to the slope surface.

(c) Anti-infiltration works to the reservoir slopes

Anti-infiltration works on the reservoir slopes to prevent the leakage water from surpass the allowable limit shall be studied. The effectiveness and economy of the anti-infiltration works shall be considered together with the protection works against wave actions, freezing-thawing effect, the foundation treatment against the piping phenomenon and the back pressure caused by groundwater acting from behind the anti-infiltration works.

(d) Anti-infiltration works to the reservoir bottom

There exists a thick mass of sediments of loamy soils with relatively low permeability coefficient. How to evaluate its efficiency in preventing the reservoir water from infiltrating through and how to design the anti-infiltration works to the reservoir bottom shall be studied.

(e) Shape-arrangement to the existing dams and the anti-infiltration works to them

The existing dams shall be arranged in shape according to the full water surface of the Reservoir and be provided with the anti-filtration work, which shall function as the continuous structure connected with the anti-infiltration work to the reservoir basin, on the upstream slope against the seepage.

(f) Total shape-arrangement of the Reservoir considering plans of the bottom area and the embankment.

Finally, the shape-arrangement of the reservoir shall be done considering the borrow area plan.

6-5-4 Comparative Study on the Anti-infiltration Works to the Reservoir (Including Risk Assessment for Leakage and Technical Specification of Trial Construction)

(1) Candidates of the anti-infiltration works

Followings shall be nominated as the candidates of the anti-infiltration works.

(a) Earth blanket coverage method

The slopes/bottom shall be covered by the earth blanket made of impervious soil layer spread and compacted. The sandy loam lying in the reservoir basin is applicable as the impervious soil. The drainage-cum-filter layer shall be provided with under the earth blanket; and the blanket surface must be protected by the slope protection work.

(b) Watertight asphalt concrete coating method

The slopes/bottom shall be coated by the pavement of watertight asphalt concrete. This method is similar to the asphalt facing work on the upstream slope of the fill-type dam. The drainage-cum-filter layer shall be provided with under this pavement.

(c) Polyethylene sheet (rubber sheet) coating

The slopes/bottom shall be coated by the impervious film such as low density polyethylene sheet. The edges of each sheet must be connected together to the ones of adjacent sheets by manpower using chemical agent and devices so that it is important how to manage these works and conduct the quality control of these works to avoid the damage due to faulty workmanship. The drainage-cum-filter layer shall be provided with under this coating work. It is a difficult choice for the slope protection to be provided with or not to be; if to be, probability of damages by its construction would increase; if not to be, durability of the work would decrease due to the friction caused by ceaseless wave actions.

(d) Bentonite sheet coating

The slopes/bottom shall be coated by the impervious thin mat of bentonite sheet. The connection work is easy only to overlap together each edge of adjacent sheets; and so it is important to manage the laying works and conduct quality control of these works to avoid the damage due to faulty workmanship caused by the easiness of the work. The drainage-cum-filter layer shall be provided with under this coating work; and its surface must be protected by the slope protection work.

(e) Soil-cement coverage

The downward seepage shall be constrained by the impervious coverage of soil-cement constructed on the slopes/bottom. Soil cement has a long history of being used empirically for small-scale waterway constructions, ground improvement works and so on but has rare example of being used as an anti-infiltration work to wide area. The drainage layer shall be provided with under this coverage; but the slope protection work is not necessary.

(f) Blanket coverage by the compacted layer of soil and bentonite-powder mixture

The slopes shall be covered by the compacted blanket layer made of soil and bentonite-powder mixture. This coverage work is treated as the standard anti-infiltration method for the industrial waste disposal pond because of high reliability on its high level imperviousness. The drainage-cum-filter layer shall be provided with under this blanket; and the blanket surface must be protected by the slope protection work.

(2) Design/construction conditions and confinement of the candidates

(a) Design condition: allowable leakage quantity and the required permeability coefficient/thickness of the anti-infiltration work

In usual dams' case, foundation treatment works are done to reduce leakage quantity through their foundation; but it is impossible to shut out all the leakage so that the scale of treatment works is designed considering the allowable leakage quantity. This allowable quantity is decided empirically considering the efficiency as a reservoir and the capability or the limit of improvement of the treatment works. In Japan's case, the target of this allowable quantity is '0.05 % of the total reservoir capacity per day'. This target value shall be applied to this reservoir.

Then, Allowable quantity = total capacity of the reservoir 94,000,000 $m^3 \times 0.0005$

$= 47{,}000\ m^3/day$

When assuming the reservoir to have the area of 9,400,000 m^2 and the average depth of 10 m,

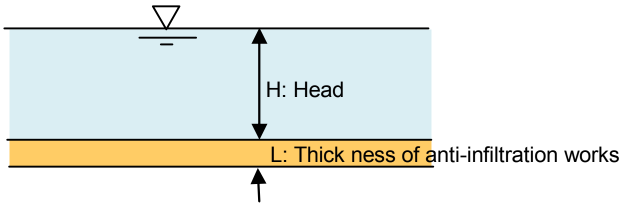
Allowable quantity per square meter = 0.005 $m^3/day/m^2$

When assuming the reservoir to have the area of 6,267,000 m^2 and the average depth of 15 m,

Allowable quantity per square meter = 0.0075 $m^3/day/m^2$

The quality of the anti-infiltration work means its permeability coefficient. The seepage direction through the reservoir bottom is vertical. In the vertical seepage problem, seepage quantity is almost decided by the layer with the lowest permeability coefficient, i.e. the anti-infiltration work. The seepage quantity through the anti-infiltration work shall be estimated by the following formula.

$$Q=k \cdot i \cdot A$$



Here, Q ; seepage quantity
k ; permeability coefficient
i ; hydraulic gradient・・・・i=H/L
A ; seepage area

The permeability coefficients and the thicknesses required for the anti-infiltration works to satisfy the allowable leakage quantity are estimated as shown in Table 6-5-4.1.

Table 6-5-4.1 Quality and Thickness Required for the Anti-infiltration Work

Reservoir model	Allowable Q (m3/day/m2)	H (m)	A (m2)	k (cm/sec)	k (m/day)	L (cm)
A=9,400,000 m2 Av. Depth=10m	0.005	10.0	1.0	5.E-05	4.E-02	8640.0
	0.005	10.0	1.0	5.E-06	4.E-03	864.0
	0.005	10.0	1.0	5.E-07	4.E-04	86.4
	0.005	10.0	1.0	5.E-08	4.E-05	8.6
	0.005	10.0	1.0	5.E-09	4.E-06	0.9
A=6,267,000 m2 Av. Depth=15m	0.0075	15.0	1.0	5.E-05	4.E-02	8640.0
	0.0075	15.0	1.0	5.E-06	4.E-03	864.0
	0.0075	15.0	1.0	5.E-07	4.E-04	86.4
	0.0075	15.0	1.0	5.E-08	4.E-05	8.6
	0.0075	15.0	1.0	5.E-09	4.E-06	0.9

(b) Construction condition: strong wind

According to the observation record of wind velocity at the Yeghvard Weather Station, strong wind blows down frequently in summer and not frequently but almost always all through a year (refer to Figure 4-3-6.13, 4-3-6.14 and 4-3.6.15). Wind pressure arises when flat surface receives wind and its degree of wind pressure is estimated according to wind velocity as follows;

The force that the body placed in fluid receives is called drag; that is calculated by the following formula.

$$D=Cd \cdot A \cdot y \cdot u^2/(2g)$$

Here, D; drag (kgf)
Cd; drag coefficient, Cd=2 in case of the flat plate
A; area of the body surface (m^2)
y; density of the fluid (kg/m^3), y=1.0 kg/m^3 corresponding to air
u; velocity of the fluid
g; gravity acceleration (m/s^2), g=9.8 m/s^2

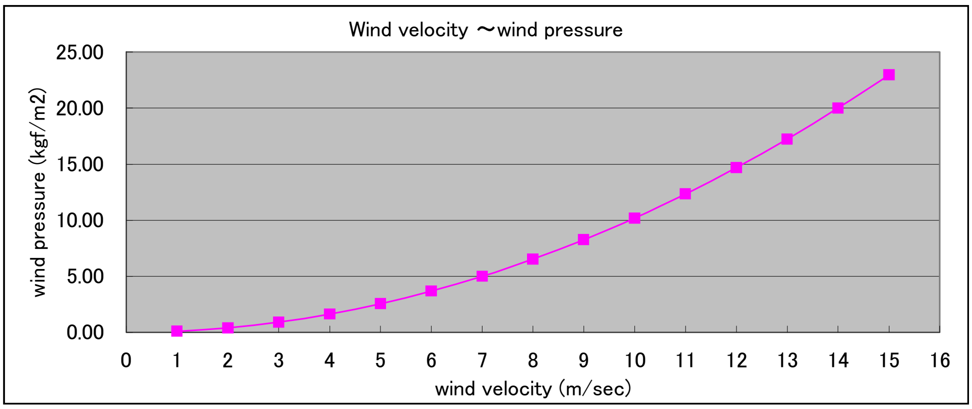


Figure 6-5-4.1 Wind Pressure Brought to a Flat Surface by Wind from in Front

The weight of sheet for anti-infiltration work is 4.9 kg in a production of polyethylene sheet 1.5 mm thick and 5.5 kg in a production of bentonite sheet 6 mm thick, so that both of them shall be blown off by 7 m/sec or 8 m/sec of wind velocity and the wind pressure brought by 10 m/sec wind to the 10 m2 (=2m x 5m) sheet reaches 100 kg which is beyond the range of manpower work.

(c) Permeability coefficient obtained/confirmed through information collection or laboratory tests

Table 6-5-4.2 shows permeability coefficient obtained in the Survey.

Table 6-5-4.2 Permeability Coefficient Obtained/Confirmed through Information Collection or Laboratory Tests

Candidate	Permeability coefficient (cm/sec)	Source
Earth blanket	5×10^{-7} (sandy loam)~3×10^{-6} (loamy sand)	Laboratory test
Asphalt concrete	1×10^{-8}	Empirically
Polyethylene sheet	1×10^{-12}	Producer's catalog
Bentonite sheet	5×10^{-9}	Producer's catalog
Soil-cement	7.7×10^{-7} ~ 3.9×10^{-8}, Sufficiency/insufficiency of curing influences the permeability.	Laboratory test
Bentonite-soil mixture	7.0×10^{-6} ~ 4.6×10^{-7}, Possibility to improve the imperviousness is left.	Laboratory test

(d) Confinement of the candidates

Table 6-5-4.3 shows confinement of the candidates for anti-infiltration works.

Table 6-5-4.3 Confinement of the Candidates for Anti-infiltration Works

Candidate	Remarks	Adopted/rejected
Earth blanket	Permeability coefficient shall be evaluated to be 5×10^{-6} cm/sec or so, then the required thickness shall be 8.64m (Table 6-5-4.1)	Too thick, rejected
Asphalt concrete	Very expensive empirically to be 150 US$/m2 or so	Too expensive, rejected
Polyethylene sheet	Very hard to execute connecting works by using chemical agents and devices under strong wind	rejected
Bentonite sheet	The required thickness shall be 9 mm (Table 6-5-4.1). Hard but possible to execute laying works due to simplicity of connecting works	Adopted as the candidate
Soil-cement	Criteria of 5×10^{-7} cm/sec shall be adequate considering the freezing/thawing effect and the differential of curing conditions between in the laboratory and in the field, and applicable to the sand-and-gravels with adjusted gradational conditions; then the required thickness shall be 86.4 cm (Table 6-5-4.1)	Adopted as the candidate
Bentonite-soil mixture	Criteria of 5×10^{-7} cm/sec shall be applicable provided farther pursuit shall be done in terms of the imperviousness improvement of bentonite sand-and-gravel mixture. The required thickness shall be 86.4 cm (Table 6-5-4.1)	Adopted as the candidate

(3) Comparative study of the anti-infiltration works

(a) Forth candidate of the anti-infiltration work

Besides the three kinds of anti-infiltration works using the materials shown above, the forth one by using the same materials shall be devised, that is the anti-infiltration work composed of two layers of soil-cement and a bentonite sheet sandwiched between them.

The bentonite sheet coverage method has disadvantages. Even if the laying works of sheets are hard but possible, it would be inevitable to meet difficulties in laying works of sheets due to strong wind.

And the appearance of faulty workmanship caused by the simplicity of the laying works shall become more frequent through the works done hastily under strong wind. These disadvantages shall be overcome by laying the sheet sandwiched between two soil-cement layers and by applying the work process as follows.

1) To prepare a foundation by soil cement to fix the sheet on it
2) To prepare a roll of sheet product
3) To prepare a heavy construction equipment not to be effected by the wind and be able to mount and spread sheet on the foundation
4) To fix the sheet quickly on to the foundation surface in such a manner as fixing it on to the soil-cement slab by driving concrete nails
5) Not to extend the sheet long but to extend the sheet short and start the fixing work from the edge toward the inner step by step.

The soil-cement coverage method has also a disadvantage; that is variation in permeability coefficient caused by non-uniformity in mixing between soil and cement and by the insufficient curing to the compacted soil-cement. This disadvantage shall be covered by the low permeability coefficient and uniformity/continuity of bentonite sheets.

This coverage method by soil-cement with a sandwiched bentonite sheet is a good measure technically in the meaning that each other's advantage covers the other's disadvantage.

(b) Thickness of the anti-infiltration work and its total structural formation

The thickness of the anti-infiltration work shall be as follows based on the evaluation in Table 6-5-4.4.

Table 6-5-4.4 Thickness of the Anti-infiltration Work

Candidate	Required thickness/ permeability coefficient (cm/sec)	Adopted
Bentonite sheet	9 mm / 5×10^{-9}	Two-ply application (6 mm×2)
Soil-cement	86.4 cm / 5×10^{-7}	90 cm
Bentonite soil mixture	86.4 cm / 5×10^{-7}	90 cm
Soil-cement with a sandwiched bentonite sheet	Soil-cement; 45 cm, bentonite sheet; one sheet Soil-cement; 5×10-7cm/sec, t=45cm⇒5×10-7cm/sec, t=45cm Bentonite sheet; 5×10-9cm/sec, t=0.6cm⇒5×10-7cm/sec, t=60cm Total; 105 cm>86.4 cm	

The anti-infiltration works must be treated together with the slope/surface protection works. The slope protection works are planned as follows according to the studies in Chapter 6-5-6.

Dam No.1, South slope ; soil-cement protection Dam No.2, North slope ; cobble-gravel rip rap

The total structural formation of each anti-infiltration work shall be planned as shown in Figure 6-5-4.2.

(c) Comparison of anti-infiltration works

Table 6-5-4.5 shows comparison of anti-infiltration works and "Soil-cement with a sandwiched bentonite sheet is selected as anti-infiltration works.

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Table 6-5-4.5 Comparison of Anti-infiltration Works

Method \ Item	Design (k: cm/sec)	A. Construction cost: item	A. Construction cost: cost	B. Construction work	C. Reliability	Judgment	A	B	C	Total
Bentonite sheet	k=5×10−9 t=6 mm	Bottom	12.6 $/m2	Frequent interruptions by strong wind	Low because of easiness of connection works done hurriedly in the strong wind condition	Bottom	10	5	3	18
		North	22.4 $/m2			North	5	5	3	13
		South	24.1 $/m2			South	5	5	3	13
Bentonite-soil mixture	k=5×10−7 t=90 cm	Bottom	18.3 $/m2	No problem	Complete enclosure is needed; if not, compscted body of bentonite- soil mixture loses its component.	Bottom	5	10	7	22
		North	28.1 $/m2			North	3	10	7	20
		South	30.4 $/m2			South	3	10	7	20
Soil-cement	k=5×10−7 t=90 cm	Bottom	15.3 $/m2	No problem	Lack of curing brings the compacted body incomplete imperviousness.	Bottom	8	10	7	25
		North	15.3 $/m2			North	9	10	7	26
		South	15.3 $/m2			South	9	10	7	26
Soil-cement with a sandwiched bentonite sheet	k=5×10−7 t=45 cm Bentonite sheet 1	Bottom	14.5 $/m2	The additional work of fixing the sheet by driving concrete nails Fewer occurrence of wind interruptions	Mistake in connection works of bentonite sheets can be covered by the continuous layer of soil-cement. Incomplete imperviousness of soil-cement can be covered by the low permeability of bentonite sheet.	Bottom	9	8	10	27
		North	14.5 $/m2			North	10	8	10	28
		South	14.5 $/m2			South	10	8	10	28
						adopted due to economy and reliability				

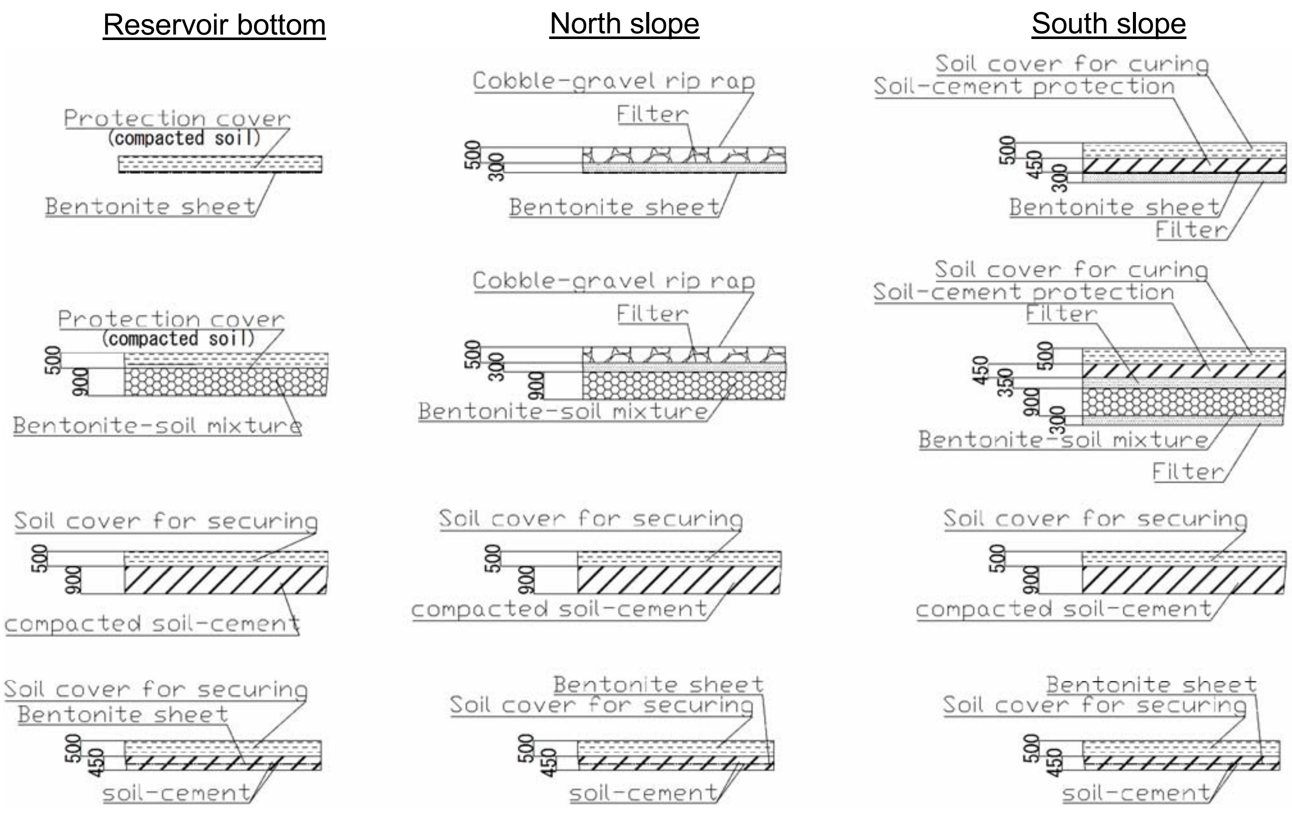


Figure 6-5-4.2 Total Structural Formation of Each Anti-infiltration Work to Each Location

(4) Risks and counter measures against leakage more than allowable volume

Although selected "Soil-cement with a sandwiched bentonite sheet" is to work as anti-infiltration works by covering disadvantage of each soil-cement and bentonite sheet by each advantage, there is a possibility that leakage volume more than allowable one will happen during operation. Therefore here examined the risks of leakage more than allowable one and countermeasures to mitigate risks.

(a) Risks

The following two (2) matters are considered as main risks for leakage more than allowable volume.

i) Higher permeability coefficient of soil cement and/or bentonite sheet than design value
ii) Cracks, gaps and spaces inside of soil-cement and/or beontonite sheet, or those boundary

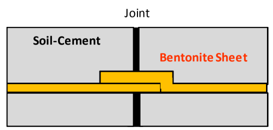
(b) Hazards of risks and counter measures against those hazards

Hazards of risks above and counter measures against those hazards are summarized as Table 6-5-4.6. Many hazards will be cleared by counter measures conducted during design, construction and construction supervise stage. However some hazards summarized in from Table 6-5-4.7 to Table 6-5-4.8 requires some tests/examinations to examine countermeasure against those.

Table 6-5-4.6 Risks of Leakage more than Allowable Volume, its Hazards and Counter Measures to Mitigate Risks

Risks				Hazards	Counter Measure				
					Design Stage	Examined through Trial Constriction		Construction Stage	Construction Supervise Stage
						Field	Laboratory		
Higher permeability coefficient than design value	Soil-Cement		Insufficient Permeability Coefficient	- Insufficient compaction - Insufficient cement volume	-	Proper - Mixing time - Water adding measure - Specification of compaction - Curing method	- Physical test - Compaction test - Permeability test	-	- High frequency of permeability Test - Abbreviated initial pounding Test targeting a completion part (not one time after completion of all the construction)
			Deterioration by freeze-thawing	- Deterioration by freeze-thawing	-			-	-
	Bentonite Sheet		Insufficient Permeability Coefficient	- Insufficient quality	-	-	- Reproduction of inspection conducted by supplier	-	- Reproduction of inspection conducted by supplier
			Efflux of Bentonite	- Breaking of upside by gravels contained in the upper layer soil-cement during construction of upper layer soil-cement	-	-	-	-	- Removal of gravels during material collection
				- Breaking of downside by protrusion by gravels contained in lower layer soil-cement	-	-	-	-	- Removal of gravels during material collection - Protrusion observation after compaction of lower layer soil-cement
			Deterioration by freeze-thawing	- Deterioration by freeze-thawing	- (Bentonite is not affected by freeze-thawing)				
Cracks, gaps, spaces	Boundary with Concrete Structure	Normal	Gap/Space	- Insufficient treatment	- Sufficient Inserting of bentonite sheet to concrete structure	-	-	-	-
	Boundary of Soil-cement and Bentonite Sheet		Gap	- Insufficient adherence of soil-cement and bentonite sheet	- (Adherence is secured by compacting soil-cement by heavy equipment)				
	Soil-Cement	Normal	Crack	- Differential settlement	- Secure capacity against difference settlement by joints	-	-	-	-
				- Bending failure by water pressure	- Stress calculation based on the results of borehole dilation test	-	- Uniaxial compression test	-	-
				- Drying shrinkage	-	- Curing method	- Drying shrinkage test	-	-
		During Earthquake		- Tensile/Compressive failure	- Secure capacity against cracks by joints	-	-	-	-
	Bentonite Sheet	Normal	Breaking	- Differential settlement	- (Bentonite sheet has some capacity against differential settlement due to its tensile strength and tenacity capacity)				
			Gap at Sheet Joint	- Insufficient treatment	- Sufficient overlapping	-	-	-	- Construction with inspectors
		During Earthquake	Breaking	- Tensile failure	- (Bentonite sheet has some capacity against deformation by earthquake due to its tensile strength and tenacity capacity)				
			Gap at Sheet Joint	- Slippage of joint	-	-	-	- Fixing bentonite sheet on the lower layer soil-cement utilizing more fasting materials than the other part. - Avoidance of stress concentrating at joints during earthquake by the arrangement of joint location*	-
Others	Soil-Cement	Normal	Wearing/Exfoliation	- Erosion at inlet/outlet point	- Consolidation around inlet/outlet point by soil-cement or concrete - Confirmation of energy dissipater efficient by hydraulic-model test	-	-	-	-

*Arrangement of joint



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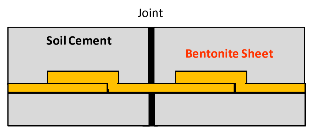

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Table 6-5-4.7 Hazards to be Examined its Mitigation Measure (Design Stage)

Target	Test	Objective	Remarks
Foundation	Borehole Dilation Test	- To calculate elastic coefficient of foundation for the examination of cracks caused by bending failure by water pressure	

Table 6-5-4.8 Hazards to be Examined its Mitigation Measure (Trial Construction - Field)

Target	Test	Objective	Remarks
Soil-Cement	Compaction Test	- To clear construction method to develop sufficient permeability coefficient such as compaction times, spreading thickness and so on	
Anti-infiltration works (Soil-Cement +Bentonite Sheet)	Abbreviated Initial Pounding Test	- To clear the notice points during construction - To confirm permeability coefficient of constructed anti-infiltration works	- Pond is constructed according to the specification cleared by compaction test.

Table 6-5-4.9 Hazards to be Examined its Mitigation Measure (Trial Construction - Laboratory)

Target	Test	Objective	Remarks
Soil-Cement	Physical Test	- To check the quality of Sand-and-Gavel Coarse and Soil-Cement	
	Compaction Test	- To check the quality of Soil-Cement - To confirm allowable time from arrangement of moisture content to casting (compacting)	
	Permeability Test	- To confirm permeability coefficient of constructed Soil-Cement	- Test is to be carried out for anti-infiltration works itself (Soil-Cement with a sandwiched Bentonite Sheet) as well
	Drying Shrinkage Test	- To confirm the possibility of cracks caused by drying shrinkage	
	Uniaxial Compression Test	- To confirm the quality of constructed Soil-Cement - To confirm uniaxial compression strength of constructed Soil-Cement for the examination of cracks caused by bending failure by water pressure	
Bentonite Sheet	Permeability Test	- To establish quality check structure during construction stage	- Reproduction of inspection conducted by supplier to confirm the permeability of produced Bentonite Sheet

Technical Specification of Trial Construction

(1) Preparation works (3 sites)

1) Location of borrow pit and trial construction yard

Based on the results of test pits survey, north-east area of reservoir is selected as borrow pit of sand-and-gravel and trial construction yards (yards are established at 3 sites). The area around TP-60 is a candidate because there are a few farm lands (see Figure-A), however actual location will be determined through discussion with PIU and local communities.

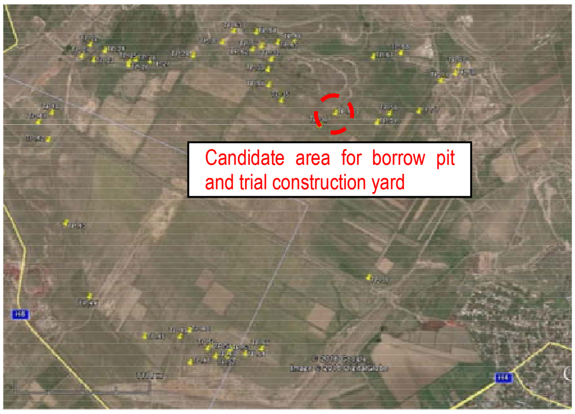


Figure-A Candidate Area for Borrow Pit and Trial Construction Yards

2) Excavation of surface soil (3 sites)

As a trial construction yard, 100m x 100m area shall be arranged. Depth of surface soil is 1.5m and total 15,000m^3 (=100m x 100m x 1.5m) of surface soil shall be removed by the bulldozer. Excavated soil shall be dumped up around the yards and utilized as cover layer for curing later.

3) Ponds excavation (3 sites)

Following the surface soil excavation, pond with bottom area 30m x 30m, depth 2.5m and slope angle 1:6.0 shall be excavated (see Figure-B). The excavated soil (sand-and-gravel) shall be sieved by the self-propelled sieving machine to adjust the gradational condition, reducing contents of the fine particle portion up to 5%. The adjusted sand-and-gravel shall be mounded as stock-pile around trial construction yard and utilized as material for soil-cement.

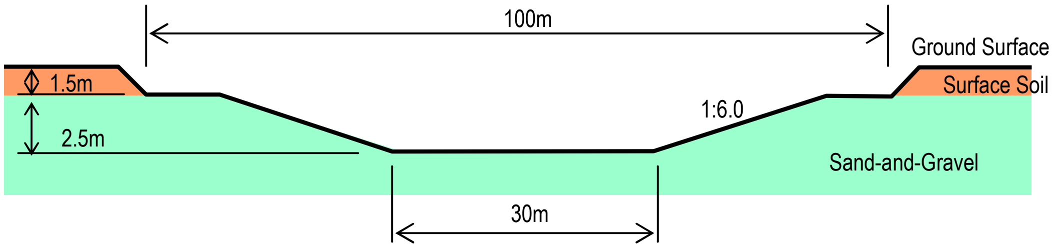


Figure-B Formation of Pond

(2) Trial compaction test to the soil-cement (1 site)

1) Objective

The objective is to decide the detail procedure of compaction work to the soil-cement with cement mixing ratio 10% and the gradational condition corresponding to “sand-and-gravel (coarse)” in the previous laboratory test, and to acquire proficiency in the laying work of bentonite-sheet.

2) Physical soil test to the materials for trial construction (1 site)

Three (3) samples shall be taken out from the sand-and-gravel stock-pile and Moisture content test, Specific gravity tests, Particle size distribution test, Atterberg limit test and Specific gravity & absorption test shall be conducted.

3) Trial mixing of sand-and-gravel with cement powder (1 site)

An inclined and rotation drum mixing machine shall be used. Through trials the adequate time of mixing and the way of adding water for the moisture content adjustment shall be decided. Adequacy shall be evaluated by uniformity; in terms of the mixing time, pH value shall be measured and in terms of the way of adding water, moisture content shall be measured. A pH meter and an electric oven shall be prepared for these measurements. Evaluation shall be made based on the variation coefficient of the measured values..

4) Standard compaction test to the mixed materials (1 site)

In case of soil-cement, it is said that the elapsed time after moisture content adjustment and mixing influences the density of the compacted layer. To grasp the degree of influence, i.e. to know how long the time to spare is till the starting of compaction, the standard compaction test shall be conducted to the five (5) samples adjusted to five (5) kinds of elapsed time.

5) Laboratory soil test to grasp the influence caused by the mixing methods

To grasp the influence caused by the differential of mixing method, Standard Compaction Test, Falling Head Permeability Test and Uniaxial Compression Test shall be conducted to the three (3) samples with 10% of cement mixing ratio and mixed by each two type of mixing machine, field and laboratory. Seven (7) days and 28 days shall be applied as the curing period of the specimens.

6) Falling head permeability test to the bentonite-sheet

Before purchasing the bentonite-sheet used for the trial construction, the detail of the testing method to evaluate the permeability coefficient at the factory shall be grasped, and the suitable permeability tests to reproduce the permeability coefficient of bentonite-sheet shall be examined.

7) Conditions of trial compaction test

i) Compaction machine: 11t vibratory roller
ii) Layer's thickness: 4 cases (15cm x 3 layers, 20cm x 2 layers, 25cm x 2 layers and 30cm x 2 layers)
iii) Passing times: 4 cases (4 vibration+2 non-vibration, 6 viv.+2 non-vib., 8 viv.+2 non-vib. and 10 viv.+2 non-vib.)

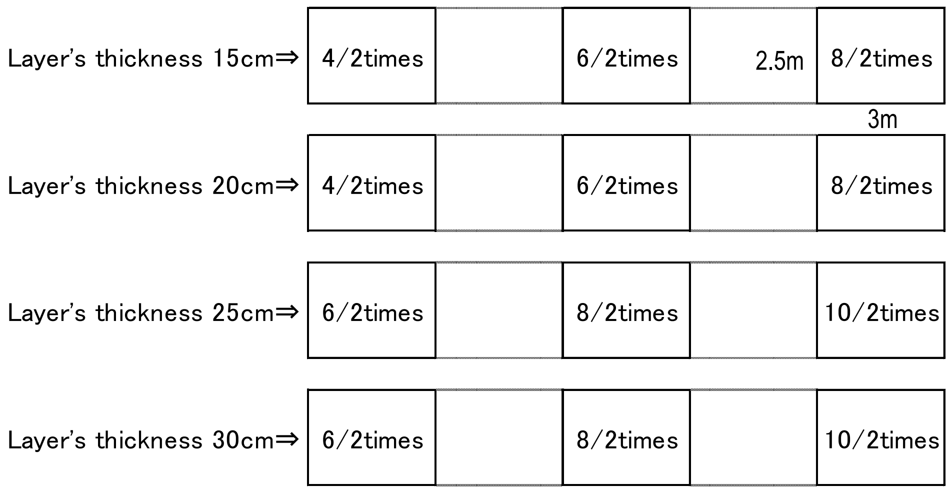


Figure-C Condition of Trial Compaction (Layer Thickness and Passing Times)

vi) Curing: 2 kinds (Soil covering curing and Sprinkle curing)

8) Process and manner of the testing

i) The trial compaction work must be carried out considering the material's property of soil-cement of which compacted density tends to be influenced by the time passage after mixing and of which permeability coefficient might be influenced much by drying due to sunshine and wind on the way of works.

ii) Immediately after the completion of the layers being compacted, soil covering and water spraying shall be provided to the layer's surface.

iii) In 7 days of curing and in 28 days of curing, sampling shall be conducted by using a core-cutter from the compacted soil-cement layer. In the laboratory, Density Test, Falling Head Permeability Test and Unaxial Compression Test shall be conducted in this order to the samples. The target of sampling is collected from both basement and covering layers.

9) Evaluation of the test result

Permeability coefficient of soil-cement shall be n x 10^{-7}cm/sec - n x 10^{-8} cm/sec.

(3) Abbreviated Initial Pounding Test (3 sites)

1) Objective

The objective is to confirm the efficiency and effectiveness of the “Compacted soil-cement layers with a sandwiched bentonite-sheet method” by constructing a considerable size of three (3) ponds.

2) Process and manner of the testing

i) The surface of ponds shall be finished by the shape arrangement work and the compaction work.

ii) On to this surface, spreading and compaction of the basement layer of soil-cement, laying work of bentonite-sheet and spreading and compaction of the covering layer of soil-cement shall be carried out in this order. The compaction work to the soil-cement layer shall be done in the manner decided through the trial compaction.

iii) Curing works selected by trial compaction test shall be provided in a short time after compaction.

vi) Joint shall be settled for each 40m at the bottom. Elastite is assumed as material to fill the joint.

v) In 28 days of curing, sampling shall be conducted by using a core-cutter from the compacted soil-cement layer. In the laboratory, Density Test, Falling Head Permeability Test and Unconfined Compression Test shall be conducted in this order to the samples. The number of sampling point shall be one (1) point per 600 m2 of the soil-cement surface; and three (3) kinds of sample shall be taken out from the one sampling point as follows;

First:	from the upper layer,
Second:	the portion containing bentonite-sheet,
Third:	from the lower layer.

vi) After finishing the sampling, the holes drilled by the core-cutter shall be buried and restored by the bentonite powder and soil-cement compaction.

vii) Water shall be led from the Arzumi-Shamiran canal and stored in the pond. The water depth in the pond shall be recorded by the water pressure meter automatically; and the evaporation depth shall be also recorded in parallel. This situation shall be kept for two months or more.

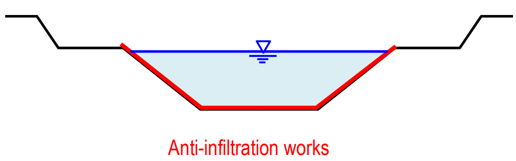
viii) In two (2) months or more, the water in the pond shall be drained and the surface of the soil-cement layer shall be exposed to sunshine to observe whether cracks appear or not on the surface.

3) Evaluation of the construction result

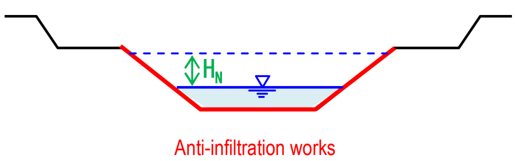
- Permeability coefficient of soil-cement shall be n x 10^{-7}cm/sec - n x 10^{-8} cm/sec.
- Permeability coefficient of anti-infiltration work shall be lower than the sufficientt one.

<u>A measure to evaluate permeability coefficient of anti-infiltration works</u>

(1) Just after pounding



(2) After N days



$$Q_N = A_{H/2} \times (H_N - E_N) = k\frac{h_{H/2}}{b} \times P_{H/2} \times N \times 864$$

$$k = \frac{A_{H/2} \times (H_N - E_N)}{\frac{h_{H/2}}{b} \times P_{H/2} \times N \times 864}$$

Where;

Q_N : Leakage volume within N days (m^3)
$A_{H/2}$: Water surface area at the level with H/2 below from original (m^2)
H_N : Water level lowering depth within N days (m)
E_N : Evaporation within N days (m)
k : Permeability coefficient of anti-infiltration works (cm/sec)
$h_{H/2}$: Average water depth at the level with H/2 below from original (m)
b : Thickness of anti-infiltration works (m)
$P_{H/2}$: Wetted perimeter at the level with H/2 below from original ((m^2)

(c) Measures in case leakage volume is more than allowable one during operation stage

Even if all the counter measures are conducted, risks cannot be cleared away completely. Therefore maintenance under precondition, there is still possibility of leakage more than allowable one, is required.

i) Measures to detect abnormal leakage volume

Measures in Table 6-5-4.9 can be considered as measures to detect abnormal leakage volume, however nothing can detect correct leakage volume.

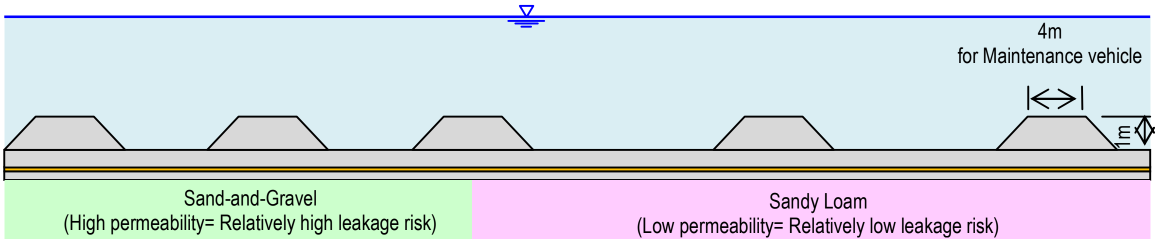
Table 6-5-4.10 Measures to Detect Leakage Volume

Measure	Evaluation
To observe fluctuation of water pressure at the back side of anti-infiltration works by pore pressure meter	It is difficult to observe fluctuation of water pressure due to un-saturation condition at the backside of anti-infiltration works.
To observe fluctuation of ground water level by monitoring wells	The fluctuation of ground water level is assumed as very little and sometimes it is difficult to judge that fluctuation is caused by leakage from reservoir or the other factors such as water from hill side, leakage from canal and so on.
To calculate leakage volume by the gap of discharge volume and fluctuation of reservoir water level	The computational error is considered as huge since reservoir volume of each elevation is very huge, fluctuation of reservoir water level is affected by evaporation and water level value observed by sensor includes margin.

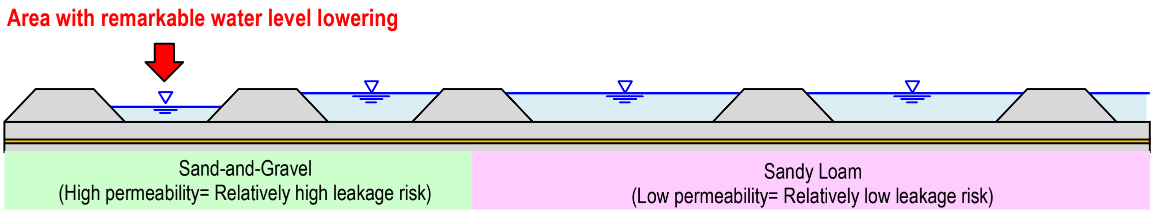
ii) Measures to detect the area of abnormal leakage and its reason

It is difficult to identify the location of abnormal leakage immediately due to wide anti-infiltration works area approx. 550ha. Therefore the following maintenance, to identify the area having relatively huge leakage and repair one after another, is recommended.

I) Division of anti-infiltration works area into small blocks by soil-cement small dike with height 1m
The area of block with sand-and-gravel basement (leakage risk is relatively high) is smaller than that with sandy loam (leakage risk is relatively low).



II) Identification of area with remarkable water level lowering after irrigation season



III) Division of remarkable water lowering area into smaller area (if necessary)

V) Field survey to grasp the reason of leakage and implementation of measures to decrease leakage according to the reason.

6-5-5 Anti-infiltration Works to the Dam Body

The anti-infiltration works to the dam body shall be given as the usual ‘core zone’ based on the following reasons.

To apply the soil-cement with a sandwiched bentonite sheet to the anti-infiltration work of the dam body would be more effective and economical than to apply the usual core zone. It would be able to say that the soil-cement with a sandwiched bentonite sheet is thin, keen to effectiveness and economical; on the other hand, the usual core zone is thick, dull to effectiveness and less economical; the problem is which is better under the consideration of safety.

The usual core zone is better even considering its disadvantage in economy; ‘thin, keen to effective’ is fragile once damaged, on the other hand ‘thick, dull in effectiveness’ is tough against damage or sustainable under a critical condition. And also it would be said that the anti-infiltration work by “core zone” is more reliable than the one by “soil-cement with a sandwiched bentonite sheet” when considering that Armenia is an earthquake country and that there is rare construction experience of concrete face rock-file dams in earthquake countries.

The dam body with the inclined core zone shall be designed in following sections.

6-5-6 Basic Design of the Dams and the Reservoir

(1) Slope protection

The slopes of the reservoir shall be protected against the erosive wave action caused by wind and also against the freezing/thawing effect in the winter season. In this section, the study shall be done in the order of the estimation of wind velocity/direction, the wave height, the requested weight of protection materials, the protection thickness against the freezing/thawing effect, and the selection of protection works and their application plan.

(a) Estimation of wind velocity/direction

1) Interview in the field

The results of the interview about wind velocity/direction to three farmers/villagers in the reservoir/town are as follows.

In what month does the strong wind blow?

Table 6-5-6.1 Answer to the Windy Month

Person	Month	Person	Month
Fa.-1	Oct. ~ Nov.	Vi.-1	Mar. Jun. Aug.
Fa.-2	Oct. ~ Nov.	Vi.-2	Nov.
Fa.-3	Sep. ~ Nov.	Vi.-3	Apr. May Jun. Jul.

How strong is it?

Table 6-5-6.2 Answer to the Wind Velocity

Wind-force class by Beaufort

Wind-force Class	Wind Name	Wind Condition/Appearance	Wind Velocity (m/sec)	Interview					
				Fa.-1	Fa.-2	Fa.-3	Vi.-1	Vi.-2	Vi.-3
0	calm	Smoke rises up streight form the chimney.	0.0～0.2						
1	light air	Wind is recognized by the smoke rising up sidling but the vane does not move.	0.3～1.5						
2	light breeze	Wind is felt on the man's face. Leaves move. The vane begins to move.	1.6～3.3						
3	gentle breeze	Leaves and thin twigs keep moving. Banners move.	3.4～5.5						
4	moderate breeze	Fugitive dust appears. Scrip rises up. Twigs move.	5.5～7.9						
5	fresh breeze	Shrub with leaves begin to move. Water surfaces of ponds have wave crests.	8.0～10.7	○	○	○	○	○	○
6	strong breeze	Big branches of trees move. Wind howls around electric cables. Hard to keep an umbrella open.	10.7～13.8						
7	near gale	Trees sway from the top to the foot. Hard to walk against the wind.	13.9～17.1						
8	gale	Twigs break off. Not able to walk against the wind.	17.2～20.7						
9	strong gale	Some of houses get damaged. Chimeys get broken and roof tiles are blown off.	20.8～24.4						
10	storm	Trees fall down by the root. Many houses get strongly damaged.	24.5～28.4						
11	violent storm	Environments, natural or artificial, are distroyed widely. Occurrence is rare.	28.5～32.7						
12	hurricane		>32.8						

Wind velocity ; mean wind velocity at the height of 10m above the ground.

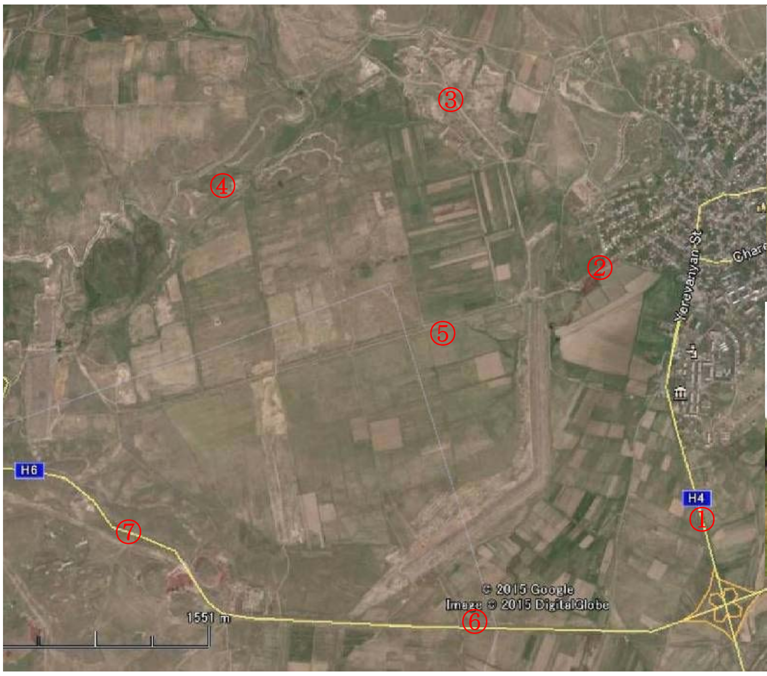
From what direction does the strong wind blow?

Table 6-5-6.3 Answer to the Wind Direction

Person	Direction	Person	Direction
Fa.-1	North	Vi.-1	North
Fa.-2	North	Vi.-2	North
Fa.-3	North	Vi.-3	North

2) Field survey of trees' inclination

The trees' trunks shall be inclined or bent to some direction due to the wind blown from a constant direction. With such expectation, the field survey was carried out; and as a result, the predominance of north or north-east in wind direction was confirmed.



①North→→→South

②South West←←←North East

③ South←←←North

④ South←←←North

⑤ North East→→→South west

⑥ South West←←←North East

⑦ South←←←North

Figure 6-5-6.1 Survey Result to the Tree Trunk's Inclination

3) Observation record in Yeghvard Weather Station

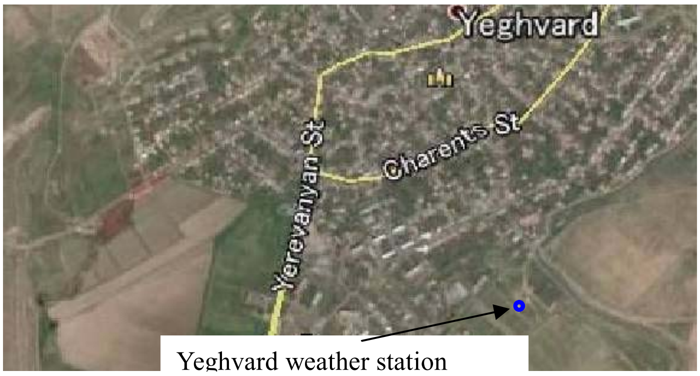


Figure 6-5-6.2 Yeghvard Weather Station, Location and Equipment

Table 6-5-6.4 Maximum Wind Velocity (m/sec)

	1	2	3	4	5	6	7	8	9	10	11	12	Year
1998	3	3	5	5	4	6	6	5	7	6	3	4	7
1999	3	5	7	10	5	8	7	5	5	4	7	2	10
2000	5	3	8	4	4	6	5	7	5	6	3	4	8
2001	8	7	8	8	7	10	11	10	10	7	8	8	11
2008	-	-	-	-	-	-	-	14	12	7	7	6	14
2009	7	8	11	14	10	11	16	14	13	8	10	6	16
2010	7	11	13	12	16	10	12	12	18	6	6	8	18
2011	5	12	11	12	10	15	13	13	13	9	12	6	15
2012	8	8	9	10	11	15	15	16	14	10	11	9	16
2013	13	6	11	10	11	12	15	15	15	11	10	9	15
2014	6	11	14	14	13	12	13	15	17	11	7	6	17

Table 6-5-6.5 Repeatability of Wind Direction and Calmness/Tranquility

Month	North	North east	East	South east	South	South west	West	North west	Tranquility
I	6	48	11	3	15	6	9	2	40
II	6	49	9	3	15	7	9	2	37
III	5	53	9	3	14	7	7	2	28
IV	5	48	7	3	17	10	8	2	23
V	5	53	7	2	15	9	7	2	22
VI	6	63	5	2	11	6	5	2	16
VII	6	73	4	1	7	4	4	1	11
VIII	6	71	4	1	9	5	3	1	11
IX	5	63	4	2	13	7	5	1	19
X	5	55	6	3	16	8	6	1	31
XI	6	47	9	3	17	9	7	2	38
XII	7	45	11	4	15	7	9	2	42
Year	6	56	7	2	14	7	6	2	27

4) Estimation of the wind velocity and the wind direction

More weight shall be given to the observation record than to the farmers'/villagers' feelings. Twenty meter per second (20 m/sec) of the maximum wind velocity shall be suitable to be adopted when considering the maximum value in recent observation records is 18 m/sec and the shortness of the observation period due to some years of missing data.

In terms of the wind direction, it would be able to consider the strong wind to blow down from north or north-east based on the inhabitant's opinion, the direction of tree trunks' inclination and the superiority in the observation record of weather station though it would be recommendable to grasp the relationship between the strong wind and the wind direction by the direct observation in future.

(b) Estimation of the wave height and the rock's weight as the slope protection work

1) Estimation of the wave height

The height of the significant wave is estimated by S.M.B. method based on the wind velocity and the blow-over distance. The wind velocity 20m/sec and the blow-over distance 3.7 km (from the north-eastern end to the south-western end of the reservoir) give the point of wave height 0.85 m.

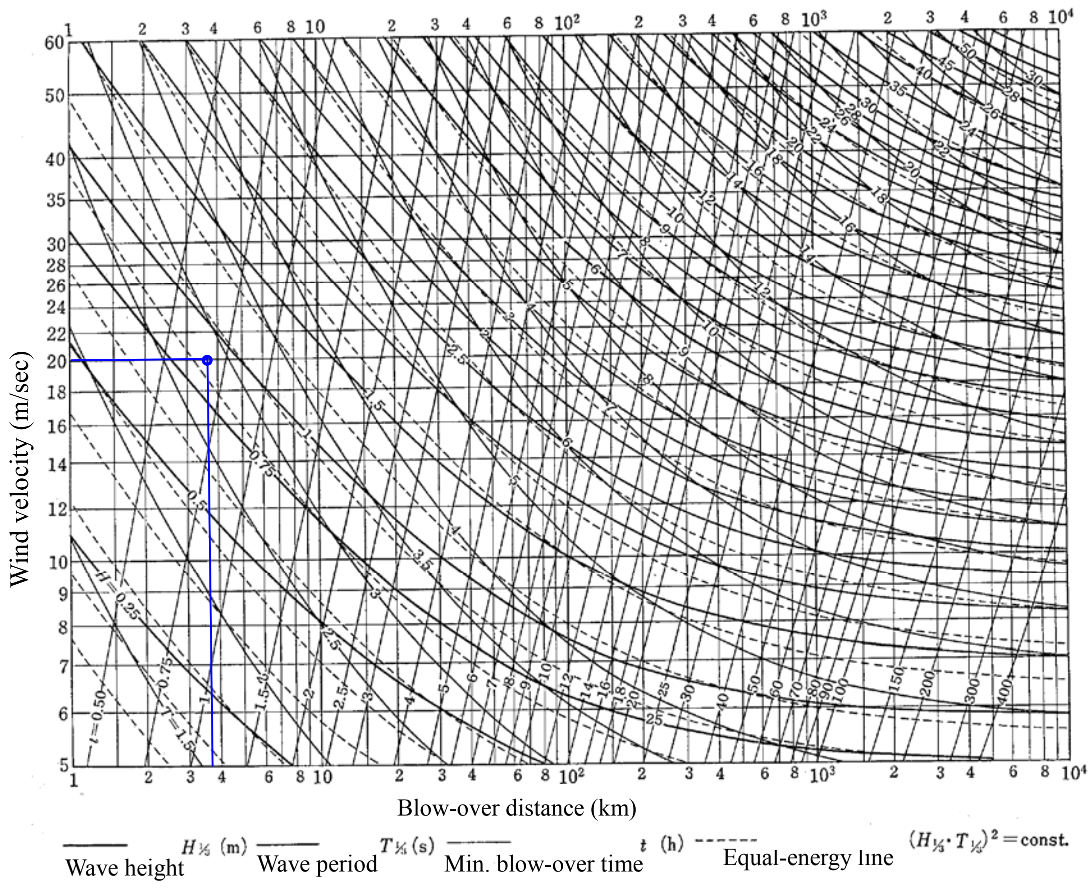


Figure 6-5-6.3 Estimation of the Significant Wave Height by SMB Method

2) Estimation of the rock's weight as the slope protection work

The rock's weight needed as the slope protection work is estimated by the Hudson's formula shown bellow.

$$W = \frac{w_r \bullet H_{1/3}^3}{K_D \bullet \left({w_r}/{w_0} - 1 \right)^3 \cot \alpha}$$

Here, W; Rock's weight (tf)

w_r; unit weight of the rock (tf/m^3)・・・・wr=2.3 tf/m^3(bulk specific gravity)

$H_{1/3}$; significant wave height ・・・・・$H_{1/3}$=0.85 m

w_0; unit weight of water (tf/m^3)・・・・・w_0=1.0 tf/m^3

α; angle between the slope surface and the horizontal line・・・cotα=3.5

K_D; coefficient to the damage percentage

Table 6-5-6.6 K_D Values to the Damage Percentage

Damage percentage	K_D
0～1 %	3.2
1～5 %	5.1
5～15 %	7.2
10～20 %	9.5
15～40 %	12.8
30～60 %	15.9

(by Hudson, 3 layers of slightly rounded rocks)

The rock's weight to the wave height $H_{1/3}$=0.85 m and the damage percentage 0～1 % (K_D=3.2) becomes 0.057 tf/m^3 as follows.

$$W = \frac{2.3 \bullet 0.85^3}{3.2 \bullet \left({2.3}/{1.0} - 1 \right)^3 \bullet 3.5} = 0.057$$

When reckoning the rock to be sphere, the grain diameter is about 40 cm as follows.

$$(4/3) \times 3.14 \times r^3 \times 2.3 = 0.057$$

$$2r = 0.36 \fallingdotseq 0.40 \text{ m}$$

(c) Protection thickness against the freezing/thawing effect

1) Thickness required for the protection coverage

The compacted soil layer shall be loosened by the repetitive action of the freezing and thawing and lose its resisting strength against shearing force so that this soil layer shall be covered by the suitable materials with the function of mitigating the conveyance of low temperature such as sand-and-gravels.

In Armenia, there is the standard in terms of the thickness of cover layer to protect the pipe from being frozen. According to this standard, 79 cm of thickness is adopted in Yeghvard area. This thickness, 80 cm rounded out from 79 cm, shall be applied also to the protection coverage over the compacted soil layer on the slopes of the reservoir and the dam body.

2) Material for the protection coverage

Table 6-5-6.7 Basis Installation Depth

The maximum depth of soil freezing		
NN	Name of station	The maximum depth, cm
1	2	3
1	Artashat	59
2	Aparan	106
3	Amasia	104
4	Ashtarak	74
5	Artik	110
6	Armavir	70
7	Aragats	77
8	Bazarchai	70
9	Berd	14
10	Verin ghukasyan	125
11	Garni	59
12	Garnahovit	109
13	Gyumri	143
14	Goris	36
15	Dilidjan	63
16	Jajur	123
17	Yerevan	60
18	Yeghvard	79
19	Yeghegnadzor	81
20	Ijevan	35
21	Qarakert	82
22	Krasnoselsk	91
23	Vanadzor	84
24	Tashir	71
25	Gavar	108
26	Kapan	14
27	Lermontov	73
28	Maralik	91
29	Martuni	116
30	Meghri	12
31	Hrazdan	96
32	Hanqavan	88
33	Spitak	103
34	Stepanavan	58
35	Sevan	114
36	Sisian	92
37	Fontan	87
38	Tsakhkadzor	115
39	Tsakhkahovit	115
40	Ararat	41
41	Odzun	42

Sand-and-gravels that lie on the slopes north-side to the reservoir shall be suitable based on the following points.

- Sand-and-gravels have the function of mitigating the conveyance of low temperature due to the existence of void air.
- The frost heaving phenomenon seldom occurs in sand-and-gravel layers.
- Even if the frost heaving phenomenon occurs, the internal friction angle of sand-and-gravels does not decrease less than its repose angle; and the repose angles of sand-and-gravels are confirmed to range from 33 degree to 41 degree in the field survey to the existing embankments.

And also, Scoria shall be suitable as a part of the protection coverage in case of Scoria being used as the buffer material between the anti-infiltration work and the slope protection work.

(d) Examination of the slope protection works and their application plan (Construction Norms IV-10.01.01-2006)

1) Candidate of the slope protection works

Rock rip rap

The rock rip rap is most common as the protection work to the upstream slope of the dam body. In this reservoir's case, this protection work shall be composed of lava rocks with the grain size of the passing percentage 50% larger than 40 cm and shall have the layer thickness of 80 cm.

And moreover, the rock rip rap shall be bedded by the 50 cm thick sand-and-gravel layer, i.e. 30 cm from 80 cm in total of the rock rip rap is assumed to be effective against freezing/thawing effect, as the anti-freezing buffer in case of the slope being provided with the soil layer of anti-infiltration work. If the impermeable liner such as the rubber sheet is provided with as the anti-infiltration work to the slope, the thickness of sand-and-gravel shall be 30 cm as the buffer material between the sheet and the rock rip rap.

Figure 6-5-6.4 Example of Rock Rip Rap

Soil-cement protection

The slope protection works by soil-cement are highly regarded in USA recently based on their performances to the big floods from 1983 to 2005 on the Santa Cruz and Rillito Rivers in Tucson, Arizona, etc. In these floods, the slope protection works by soil-cement only survived in spite of many other protection works were damaged hard or lost. (Refer to "Performance of Flood-tested Soil-cement Protected Levees, by Kenneth D. Hansen etc., 31st US Society on Dams (USSD) Conference)

6. Flow in Santa Cruz River north of Congress St. Bridge,

Figure 6-5-6.5 Flow in Santa Cruz River north of Congress St.Bridge, 1n 1993

In terms of the weathering durability of soil-cement, the performance of the US Bureau of Reclamation (USBR) soil-cement test section in the Bonny Reservoir built in 1951 provides a positive example of the one exposed long to the wave action and an average of 140 freeze-thaw cycles per year as shown in Figure 6-5-6.6.

[Extraction from the literature above]

As shown in Figure 15, taken in 2008, there has been some breakage of soil-cement layers due to poor bond between the individual 6-inch thick soil-cement layers. The lower portion of the lifts were thus subject to wave erosion due to lower cement content at the bottom of the layers that is typical of the in-place mixing method together with lower density in this area due to the compaction method.

Based on the long-term performance of exposed soil-cement constructed with adequate cement content at Bonny Reservoir and other projects subjected to freeze-thaw cycles, a small amount of deterioration may be expected. This is usually at the outer edges of layers where lower density occurs due to lack of restraint of the edge when compacted. Once this minor loose material is washed away, the soil-cement is sound and has shown very fine weathering durability.

Figure 6-5-6.6 Soil-cement Slope Protection

And the test results of freezing/thawing test, slaking test and sodium sulfate soundness test conducted in this Survey indicate high durability of soil-cement against weathering (refer to Chapter 4-3-5 (3)).

An advantage of the slope protection work by soil-cement is that this can function not only as the protection against wave erosive actions but also as the protection coverage against the freezing/thawing effect. 45 cm in thickness shall be given to this protection work from the view point of the weight needed to wave actions and 35 cm thick buffer/filter layer shall be provided with between the upper protection and the lower anti-infiltration work; then total thickness of 80 cm functions against the freezing/thawing effect.

Cobble-gravel rip rap

The area of hilly slopes north-side to the reservoir produces sand-and-gravels from which cobble-and-gravels for the rip rap use shall be obtained through screening. An advantage of this material is that the layer can function not only as the protection against wave actions but also as the coverage against the freezing/thawing effect. In addition, the construction easiness of the materials being spread and compacted layer by layer on the slope surface is also a big advantage.

Figure 6-5-6.7 Example of Cobble-Gravel Rip Rap

But this type of protection work is applicable only to the north and the east slopes where wave actions are little because the grain size/weight of cobbles is not enough to stand wave actions on the slopes on the lee.

2) Selection of slope protection works and their application plan

Table 6-5-6.8 Selection of Slope Protection Works and their Application Plan

Slope / Protection work	Dam No.1		Dam No.2		North slope		South slope	
	Wave action hard	Freezing-thawing	Wave action not hard	Freezing-thawing	Wave action not hard	Freezing-thawing	Wave action hard	Freezing-thawing
Rock rip rap	work	not work	work	not work	work	not work	work	not work
Cobble-gravel rip rap	not work	work	work	work	work	work	not work	work
Soil-cement	work	work	work	work	work	work	work	work
Adoption	Soil-cement		Cobble-gravel rip rap (due to economy)		Cobble-gravel rip rap (due to economy)		Soil-cement	

- convergence of wind direction⇒hard wave action to Dam No.1 & south slopes
- rock rip rap⇒work against hard wave action, not work against freezing-thawing due to large void
- cobble-gravel rip rap⇒not work against hard wave action, work against freezing-thawing
- soil-cement⇒work against hard wave action and freezing-thawing
- economy; Cobble-gravel rip rap < Soil-cement < rock rip rap (refer to Table 6-5.6.9~ 6-5.6.11)
  (4.5 US$/m^2) (8.6 US$/m^2) (9.8 US$/m^2)

Table 6-5-6.9 Cost Estimation of Cobble-gravel Rip Rap (per 1,000 m^2 of Construction)

No.	BOQ	Normative base	Description of works	Measurement unit	Quantity	Unit price AM drams		Unit price of works, AM drams	Overall sum, AM drams
						Salary AM drams	Machinery operations AM drams		
1	1.1	**E1-1610**	Excavation by bulldozer 96Kvt or 130 hourse-power, replacing excavated soil up to 10m, soil grade I	1000m3	0.0472		57,070	57,070	2,694
2	1.1	**E1-1617**	Additional 50m replacment by bulldozer 96Kvt or 130 hourse-power, soil grade I	1000m3	0.0472		252,739	252,739	11,929
3	1.2	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.0472	6,362	157,948	164,310	7,755
4	1.3	**C310-1-1**	Transportation of soil up to 1 km	ton	40		734	734	29,438
5	2.1	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3	0.566	9,720	240,917	250,637	141,861
6	2.2	**C310-1-1**	Transportation of soil up to 1 km	ton	283		734	734	207,653
7	2.3	**331430**	Work of sieving machine	machine/ hour	6.90		4,757	4,757	32,817
8	2.4	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3	0.3	9,720	240,917	250,637	75,191
9	2.5	**C310-2-1**	Transportation of clay-sandy soil up to 2 km	ton	585		897	897	524,637
10	2.6+2.7	**E36-5**	Construction of upper part of the dam core and screen, any soils except rocks, Compactor capacity-heavy	1000m3	0.3	33,186	337,257	370,442	111,133
11	3.1	**E1-1610**	Excavation by bulldozer 96Kvt or 130 hourse-power, replacing excavated soil up to 10m, soil grade I	1000m3			57,070	57,070	
12	3.1	**E1-1617**	Additional 50m replacment by bulldozer 96Kvt or 130 hourse-power, soil grade I	1000m3			252,739	252,739	
13	3.2	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3		6,362	157,948	164,310	
14	3.3	**C310-1-1**	Transportation of soil up to 1 km	ton			734	734	
15	4.1	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3	0.5	9,720	240,917	250,637	125,319
16	4.2	**C310-1-1**	Transportation of soil up to 1 km	ton	975		734	734	715,413
17	4.3		Work of sieving machine	machine/ hour			4,757	4,757	
18	4.4	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3		9,720	240,917	250,637	
19	4.5	**C310-2-1**	Transportation of clay-sandy soil up to 2 km	ton			897	897	
20	4.6+4.7	**E36-5**	Construction of upper part of the dam core and screen, any soils except rocks, Compactor capacity-heavy	1000m3	0.5	33,186	337,257	370,442	185,221
			Total						**2,171,061**
									4,511 US$

Table 6-5-6.10 Cost Estimation of Soil-cement Coverage (per 1,000 m² of Construction)

No.	BOQ	Normative base	Description of works	Measurement unit	Quantity	Unit price AM drams						Unit price of works, AM drams	Overall sum, AM drams
						Salary AM drams	Machinery operations AM drams			Prices			
								Measurement unit	Demand for initial unit of work	Material unit price, AM drams	Overall price for initial unit of work, AM drams		
1	1.1	**E1-1610**	Excavation by bulldozer 96Kvt or 130 hourse-power, replacing excavated soil up to 10m, soil grade I	1000m3	0.0944		57,070					57,070	5,387
2	1.1	**E1-1617**	Additional 50m replacment by bulldozer 96Kvt or 130 hourse-power, soil grade I	1000m3	0.0472		252,739					252,739	11,929
3	1.2	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.0472	6,362	157,948					164,310	7,755
4	1.3	**C310-1-1**	Transportation of soil up to 1 km	ton	40		734					734	29,438
5	2.1	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3	0.583333	9,720	240,917					250,637	146,205
6	2.2	**C310-1-1**	Transportation of soil up to 1 km	ton	569		734					734	417,325
7	2.3	**331430**	Work of sieving machine	machine/ hour	7.11		4,757					4,757	33,822
8	2.4	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3	0.35	9,720	240,917					250,637	87,723
9	2.5	**C310-2-1**	Transportation of clay-sandy soil up to 2 km	ton	683		897					897	612,076
10	2.6+2.7	**E36-5**	Construction of upper part of the dam core and screen, any soils except rocks, Compactor capacity-heavy	1000m3	0.35	33,186	337,257	m3	100			370,442	129,655
11	3.1	**E1-1610**	Excavation by bulldozer 96Kvt or 130 hourse-power, replacing excavated soil up to 10m, soil grade I	1000m3	0.0499		57,070					57,070	2,848
12	3.1	**E1-1617**	Additional 50m replacment by bulldozer 96Kvt or 130 hourse-power, soil grade I	1000m3	0.0499		252,739					252,739	12,612
13	3.2	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.0499	6,362	157,948					164,310	8,199
14	3.3	**C310-1-1**	Transportation of soil up to 1 km	ton	42		734					734	31,122
15	4.1	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.5625	6,362	157,948					164,310	92,425
16	4.2	**C310-1-1**	Transportation of soil up to 1 km	ton	478		734					734	350,828
17	4.3	**331430**	Work of mixing machine	machine/ hour	1.91		8,787					8,787	16,806
18	4.4	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.45	6,362	157,948					164,310	73,940
19	4.5	**Market**	Cement price with transportation	ton	36			ton	1	41,667	47,542	47,542	1,711,500
20	4.7	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.45	6,362	157,948					164,310	73,940
21	4.9+4.1	**E27-5-1 x 3 layers**	Soil-Cement Mixture spreading, compacting, curing, Pabble - crushed stone - sandy fine soil, Layer thickness 150mm	100m2	10	7,364	21,605					28,969	289,687
			Total										**4,145,221**
			including										**8,613 US$**

Table 6-5-6.11 Cost Estimation of Rock Rip rap (per 1,000 m^2 of Construction)

No.	BOQ	Normative base	Description of works	Measurement unit	Quantity	Unit price AM drams						Unit price of works, AM drams	Overall sum, AM drams
						Salary AM drams	Machinery operations AM drams	Prices of materials					
								Description of materials	Measurement unit	Demand for initial unit of work	Prices Material unit price, AM drams		
1	1.1	**E1-1610**	Excavation by bulldozer 96Kvt or 130 hourse-power, replacing excavated soil up to 10m, soil grade I	1000m3	0.0472		57,070					57,070	2,694
2	1.1	**E1-1617**	Additional 50m replacment by bulldozer 96Kvt or 130 hourse-power, soil grade I	1000m3	0.0472		252,739					252,739	11,929
3	1.2	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.0472	6,362	157,948					164,310	7,755
4	1.3	**C310-1-1**	Transportation of soil up to 1 km	ton	40		734					734	29,438
5	2.1	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3	0.833	9,720	240,917					250,637	208,781
6	2.2	**C310-1-1**	Transportation of soil up to 1 km	ton	708		734					734	519,537
7	2.3	**331430**	Work of sieving machine	machine/ hour	8.85		4,757					4,757	42,106
8	2.4	**E1-1563**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil III	1000m3	0.5	9,720	240,917					250,637	125,319
9	2.5	**C310-2-1**	Transportation of clay-sandy soil up to 2 km	ton	850		897					897	762,292
10	2.6+2.7	**E36-5**	Construction of upper part of the dam core and screen, any soils except rocks, Compactor capacity-heavy	1000m3	0.5	33,186	337,257	Water	m3	100		370,442	185,221
11	3.1	**E1-1610**	Excavation by bulldozer 96Kvt or 130 hourse-power, replacing excavated soil up to 10m, soil grade I	1000m3	0.0425		57,070					57,070	2,425
12	3.1	**E1-1617**	Additional 50m replacment by bulldozer 96Kvt or 130 hourse-power, soil grade I	1000m3	0.0425		252,739					252,739	10,741
13	3.2	**E1-1561**	Excavtion by excavator on crawler, bucket capacity 2,5 - 3m3, loading excavated soil to dump track, grade of soil I	1000m3	0.0425	6,362	157,948					164,310	6,983
14	3.3	**C310-1-1**	Transportation of soil up to 1 km	ton	37		734					734	27,287
15	4.1	**E3-106**	Loosening of V grade soil by blasting with blasthole charges using rotary-percussion drilling machin, hummer dimameter 105mm	$100m^3$	2	9,229	37,775	Drilling crowns	item	0.09	3,120	99,322	158,915
16	4.2	**60234+140551**	Hydro-Hummer work , which is attached to excavator	machine/ hour	0.48		17,312					17,312	8,310
17	4.3	**E1-1541**	Excavtion by excavator on crawler or wheeled, bucket capacity 0,8-1m3, dumping excavated soil a side, grade of soil V	1000m3	0.8	12,783	560,917					573,700	458,960
18	4.4	**C310-5-1**	Transportation of clay-sandy soil up to 5 km	ton	1,440		1,359					1,359	1,956,686
19	4.5	**E38-1**	Rockfill in dam body, Thickness of layer under 1m	1000m3	0.8	13,235	225,454	Water	m3	300		238,689	190,951
			Total										**4,716,331**
													9,800 US$

(2) Dam crest protection

The dam crest shall be protected against the freezing and thawing effect and also against the damage caused by the wheel load or friction of vehicles.

In Armenia, in the area around Yerevan, roofs of residential houses are made of concrete with a 25 cm thick heat-insulating layer of coarse Scoria between the outer slab and the inner slab. According to this manner, a 25 cm thick Scoria layer shall be provided to the crest as the protection against the freezing and thawing effect.

Over this Scoria layer, 30 cm thick sand-and-gravel layer shall be provided as the protection against the vehicles' wheels. This sand-and-gravel layer shall have the supplemental effect to the heat-insulating function of the Scoria layer.

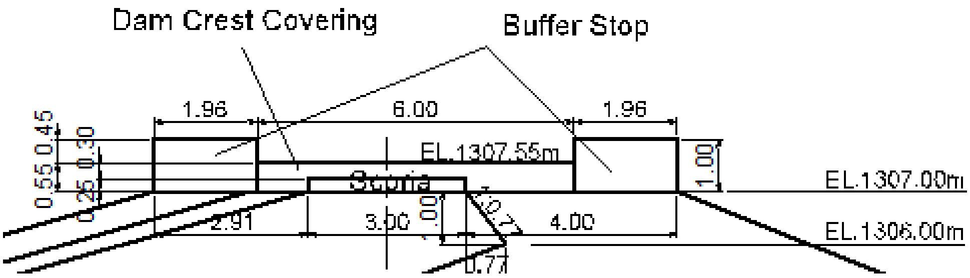


Figure 6-5-6.8 Illustration of the Dam Crest Protection

(3) Freeboard elevation of the dam body

(a) Applied standard and the calculation formula

The freeboard elevation of the dam body is given by the following formula.

$$H_{fr}=H_f+h_w+h_c+1 \text{ (in case of } h_w+h_c<1.0,\ H_{fr}=H_f+2.0)$$

Here, H_{fr}; Freeboard elevation of the dam body

H_f ; Full water surface elevation in the reservoir ・・・H_f=E.L. 1,305.0 m (Reservoir plan)

h_w ; Height of wave run-up

h_c ; Height of wave caused by an earthquake

* The reservoir is not provided with a spillway so that the freeboard elevation is decided to the full water surface.

(b) Height of wave run-up

The calculation formula of wave run-up is shown by Van der Meer and Janssen in their work "Wave run-up and wave overtopping at dikes, 1995" as follows.

General formula ; $R_{u2\%}/H_{1/3}=1.6\gamma_b\gamma_f\gamma_\beta\xi_{op}$

$$\xi_{op}=\tan\alpha/(s_{op})^{1/2}$$

To the rock slopes ; for $\xi_{op}<1.5$ ・・・ $R_{u2\%}/H_{1/3}=0.88\xi_{op}$

for $\xi_{op}>1.5$ ・・・ $R_{u2\%}/H_{1/3}=1.1(\xi_{op})^{0.46}$

Here,

$R_{u2\%}$; Wave run-up
$H_{1/3}$; significant wave height ($H_{1/3}$ =0.85, from the examination of slope protection)
ξ_{op} ; breaker parameter
γ_b ; reduction factor for a berm (in case of straight slope, γ_b=1.0)
γ_f ; reduction factor for slope roughness (in case of smooth slope, γ_f =1.0)
γ_β; reduction factor for oblique wave attack (in case of perpendicular wave, γ_β=1.0)
α ; slope angle (now, tanα=1/3.5)
s_{op} ; wave steepness (on the slope gentler than 1/3, wave breaks. At the moment of wave breaking, wave steepness s_{op} is 1/7.)

Then, in case of smooth slope ; $\xi_{op}=\tan\alpha/(s_{op})^{1/2}=(1/3.5)/(1/7)^{1/2}=0.76$

$$R_{u2\%}=1.6\gamma_b\gamma_f\gamma_\beta\xi_{op}\, H_{1/3}=1.6\times1.0\times1.0\times1.0\times0.76\times0.85=1.03\text{ m}$$

in case of rock slopes ; $R_{u2\%}=0.88\xi_{op}\, H_{1/3}=0.88\times0.76\times0.85=0.57$ m

(c) Wave height caused by an earthquake

The calculation formula of wave height caused by an earthquake is shown below.

$$h_c=\frac{1}{2}\bullet\frac{k\bullet\tau}{\pi}\bullet\sqrt{g\bullet H_0}$$

Here, h_c ; wave height caused by earthquake (m)
k ; earthquake coefficient (now, k=0.12)
τ; seismic wave cycle (usually τ=1.0 second is applied)
H_0 ; water depth in the reservoir at the time of full water level (H_0=16 m)
g ; acceleration of gravity (g=9.8 m/s^2)

Then, $h_c=\frac{1}{2}\bullet\frac{k\bullet\tau}{\pi}\bullet\sqrt{g\bullet H_0}=\frac{1}{2}\bullet\frac{0.12\bullet1.0}{3.14}\bullet\sqrt{9.8\bullet16.0}$ =0.24 m

(d) Freeboard elevation of the dam body

In case of smooth slopes ; $h_w+h_c=1.03+0.24=1.27>1.0$
$H_{fr}=H_f+h_w+h_c+1$=E.L.1,305.0+1.03+0.24+1=EL.1,307.27 m

In case of rock slopes ; $h_w+h_c=0.57+0.24=0.81<1.0$
$H_{fr}=H_f+2.0$= E.L.1,305.00+2.0=E.L.1,307.00 m

[Dam-No.1]

The soil-cement slope protection is applied to the upstream slope of Dam No.1. The upper portion beyond the full water level shall be constructed layer by layer spread horizontally considering the slope to be provided with steps, the surface of the slope protection not to become smooth and maintenance workers not to slip down into the reservoir. Then, the elevation E.L.1,307.00 shall be applied to the freeboard elevation here.

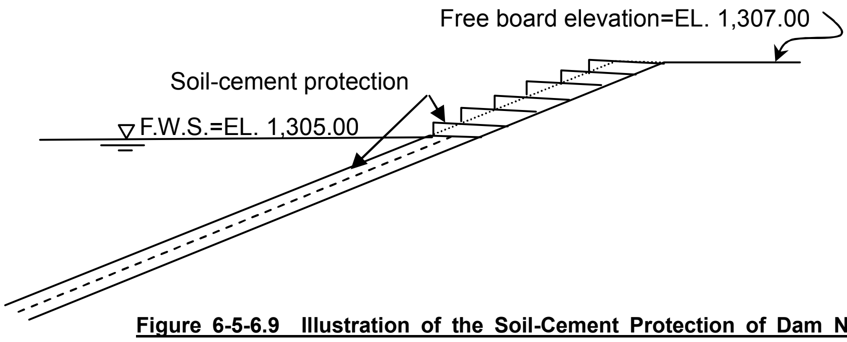


Figure 6-5-6.9 Illustration of the Soil-Cement Protection of Dam No.1

[Dam-No.2]

The cobble-gravel rip rap protection is applied to the slope of Dam No.2 which is treated as the rock slope so that the elevation E.L.1,307.00 shall be given as the freeboard elevation.

(4) Dam crest elevation

The dam crest elevation can be given by adding the dam crest protection thickness to the freeboard elevation of the dam body.

Then, Dam crest elevation = Freeboard elevation + Crest protection thickness

= E.L. 1,307.0 + 0.55
= EL. 1,307.55

(5) Typical cross-section of dams

1) Dam type and Zoning

During the Soviet era, a part of dam body (a part of Sand-and-Gravel zone) was constructed. According to the results of field surveys, these existing dam bodies have enough strength and it is judged these existing dam body can be one part of newly constructed dam bodies. Therefore, for the effective use of existing dam bodies, inclined core type is selected for both Dam No.1 and Dam No.2.

Since Dam No.1 does not have enough height, Sand-and-Gravel zone is newly constructed on existing dam body and Core zone (impervious zone) is constructed at the surface of upstream side. On the other hand, Dam No.2 has enough height then only Core zone for upstream surface is required.

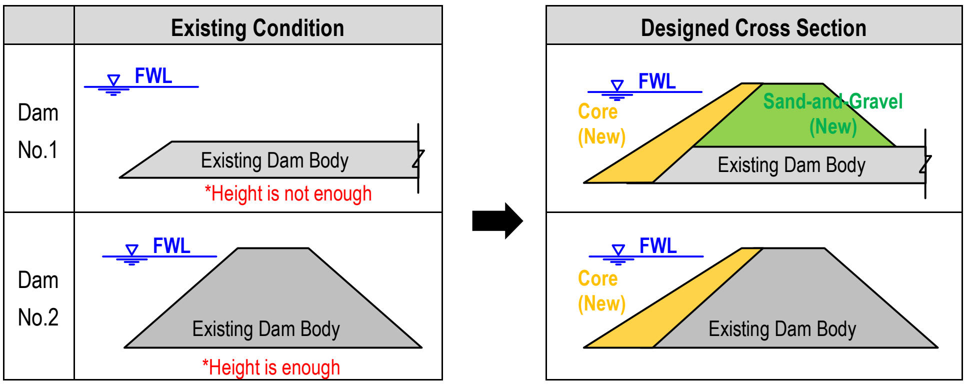


Figure 6-5-6.10 Outline of Designed Cross Section

Core zone is arranged with its minimum width more than 50% of head as shown in the Figure 6-5-6.11. Due to this condition and effective use of existing dam bodies, slope angle of upstream 1:3.5 and downstream 1:2.75 are selected.

Filter zone is arranged in front of core zone to prevent Core zone material to efflux and filter zone is protected by slope protection.

Figure 6-5-6-12 show typical cross section designed according to the conditions above.

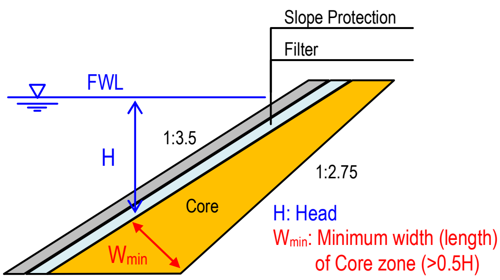


Figure 6-5-6.11 Arrangement of Core Zone

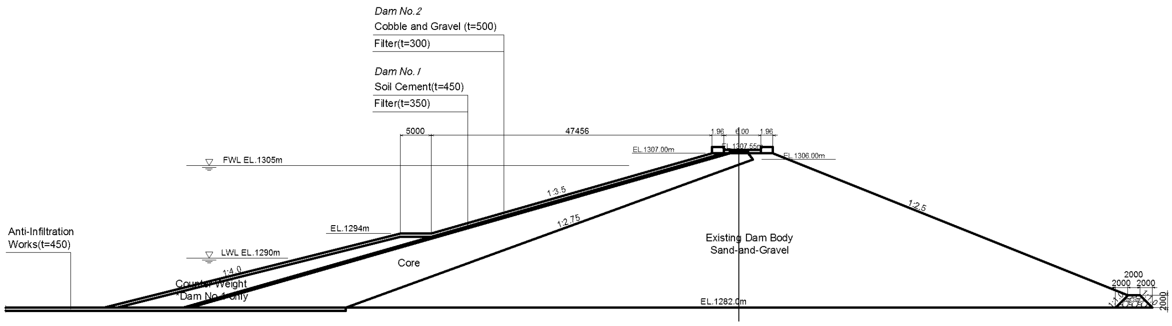


Figure 6-5-6.12 Typical Cross Section

3) Stability analysis

i) Required safety factor

Safety factor Fs is determined according to the formula below.

$$Fs = \frac{R}{\Sigma(\gamma_f F)} \geq \gamma_{lc} \times \gamma_n$$

$$(Required\ Safety\ Factor) = \gamma_{lc} \times \gamma_n = 1.0 \times 1.25 = 1.25$$

Where;

R: Bearing capacity

F: Force Factor

γ_f: Loading reliability coefficient (=1.0 in normal condition, 0.95 in earthquake condition)

γ_{lc}: Loading combination coefficient (=1.0)

γ_n: Reliability coefficient by structure(=1.25)

Table 6-5-6.12 Reliability Coefficient by Structure γn

Dam Class	I	II	III	IV
γ_n	**1.25**	1.20	1.15	1.10

Dam Classification

Criteria to determine the class of dam are shown in Table 6-4-5.13 to 6-4-5.16. and results of checking condition of Yeghvard reservoir against these criteria are shown as below. As a result, Yeghvard reservoir is classified as **Class-I.**

Criterion-1: Dam height
25.55m with rock foundation → **Class III**

Criterion-2: Social-Economic Responsibility
Dam capacity= 94 MCM → **Class III**
Irrigation area= 13,000ha → **Class IV**

Criterion-3: Protective Structures
-

Criterion-4: Consequence of possible accidents
Number of injured person -
Number of affected person -
Economic Damage -
Areas=2Marzs → **Class I**

-: Not identified in this Survey

Table 6-5-6.13 Criterion-1: Dam Height

Structure	Base Soil Type*	Class I	Class II	Class III	Class IV
Earth Fill Dam	A	More than 80 m	From 50 to 80m	From 20 to 50m	Less than 20 m
	B	More than 65 m	From 35 to 65m	From 15 to 35m	More than 15 m
	C	More than 50 m	From 25 to 50m	From 15 to 25m	More than 15 m

*A: Rock, B: Solid or Semi-Solid sand, C: Coarse and clay

Table 6-5-6.14 Criterion-2: Social-Economic Responsibility

	Class I	Class II	Class III	Class IV
Dam Capacity (MCM)	More than 1,000	From 200 to 1,000	From 50 to 200	Less than 50
Irrigation Area (thousands ha)	More than 300	From 100 to 300	From 50 to 100	Less than 50

Table 6-5-6.15 Criterion-3: Protective Structures

*Estimated submerge depth (m)

		Class I	Class II	Class III	Class IV
Housing Density at Downstream Side (m^2/ha)	More than 2,500	More than 5	More than 5	More than 3	-
	From 2,100 to 2,500	More than 8	More than 8	More than 5	More than 2
	From 1,800 to 2,100	More than 10	More than 10	More than 8	More than 5
	Less than 1,800	More than 15	More than 15	More than 10	More than 8
Leisure, Health and Sanitation Structure		-	More than 15	More than 15	More than 10
Industrial organizations (MAS*/year)	More than 50	More than 5	Up to 3	Up to 2	-
	From 10 to 50	More than 8	Up to 5	Up to 3	Up to 2
	Less than 10	More than 8	Up to 8	Up to 5	Up to 3
Cultural and Natural Monuments		More than 3	Up to 3	-	-

*MAS: Minimal Amount of Salary

Table 6-5-6.16 Criterion-4: Consequences of Possible Accident

	Class I	Class II	Class III	Class IV
Number of Inhabitants Who Will Be Injured by the Accident (persons)	More than 3,000	From 500 to 3,000	Up to 500	-
Number of Peopled Whose Living Conditions Will Be Affected by the Accident (persons)	More than 20,000	From 2,000 to 20,000	Up to 2,000	-
Possible Economic Damage (MAS*1)	More than 50	From 10 to 50	From 1 to 10	Less than 1
Areas where will be emergency situation by the accident*2	Within two or more Marzs, or territory of neighboring country	Within one Marz or two or more formations of neighboring country	Within one Marz	Within one Marz

*1: MAS: Minimal Amount of Salary
*2: The collaption of dam causes the over flow of Kasakh River for both Kotayk Marz and Aragatsotn Marz side.

ii) Analysis case

Two (2) analysis cases are selected taking into consideration of the combination of water level and PGA coefficient k shown in Table 6-5-6.17. Analysis case in case flood is not necessary because all the water flowing into reservoir is controlled one at intake point of Hrazdan river and at the inlet of feeder canals, and flood never flow into the reservoir.

Table 6-5-6.17 Analysis Cases

Case		Water Level	k	Required Safety Factor Fs
Case-1	Normal Condition with maximum scale earthquake	FWL EL.1305m	0.12	1.25
Case-2	Sudden water lowering with half sale earthquake	FWL EL.1305m ➔ LWL EL.1290m	0.06	1.25

iii) Physical Property

Physical properties utilized for stability analysis are shown in the Table 6-5-6.18 (determination of these values is described in the Appendix J).

Table 6-5-6.18 Physical Properties for Stability Analysis

Zone	Wet Density γ_t (kN/m^3)	Saturated Density γ_{sat} (kN/m^3)	Cohesion c (kN/m^2)	Internal Friction Angle φ (Degree)
1. Core	18.99	19.19	21.40	24.30
2. Filter	19.25	20.00	0	38.00
3. Existing Dam Body*	19.30	19.97	0	38.00
4. Slope protection	22.00	22.00	0	38.00
5. Dam Crest Covering	19.30	19.97	0	33.00
6. Counter Weight	19.30	19.97	0	33.00

* Same values are applied to Sand-and-Gravel zone

iv) Results of analysis

Stability analysis is conducted by Armenian method and Japanese method, and results are shown in Table 6-5-6.19. According to the results, calculated safety factor is more than required one by Japanese method but less than in Armenian method. Also the calculated safety factor is quite different.

Table 6-5-6.19 Results of Stability Analysis (Calculated Safety Factor)

	Armenian Method				Japanese Method			
	Upstream slope		Down Stream Slope*		Upstream slope		Down Stream Slope*	
Case-1	0.85 < 1.25	NG	0.70 < 1.25	NG	1.44 > 1.25	OK	1.43 > 1.25	OK
Case-2	1.13 < 1.25	NG	-		1.26 > 1.25	OK	-	

*Sine it is clear that calculated safety factor of case-2 is more than case-1, the calculation of case-2 is omitted.

The reasons why the calculated safety factors has such difference are considered to be caused by the following two matters.

a) Increasing ratio of PGA coefficient k

In case earthquake happens, acceleration of dam crest is higher than that of bottom. Figure 6-5-6.13 shows the vertical distribution of acceleration. k is PGA coefficient and k_Y is acceleration coefficient at the point Y m below from dam crest.

Figure 6-5-6.13 shows vertical distribution of acceleration increase ratio (=k_Y/k) in this Survey and determined in Japanese standard. Additionally, distribution from the results of FEM analysis and observed data of dams in Japan are shown as references as well. Increasing ratio of Japanese standard and references at dam crest (Y=0) is 2 or 3, however the value is 5 in this Survey, calculated value according to Armenian standard. This value is almost two times of Japanese standard.

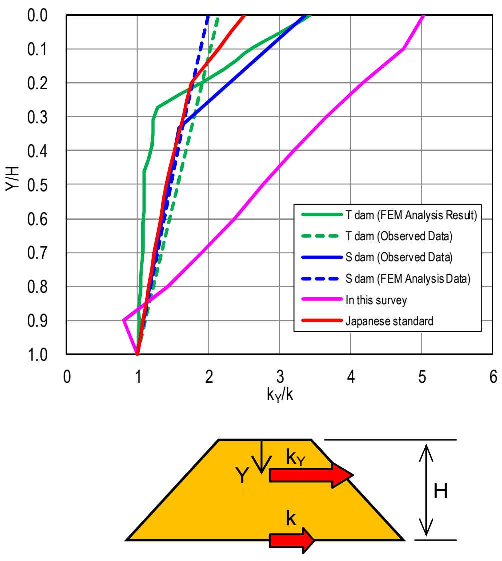


Figure 6-5-6.13 Increasing of k

In case big earthquake happens, dam body is deformed by earthquake shaking, on the other hand, dumping ratio of soil materials becomes bigger as the deformation becomes big. In this case, acceleration increasing ratio at dam crest is not so high.

It is supposed that the effect of this damping ration is not considered well in Armenian standard.

Since dams designed using this Japanese standard has no experience of collapse by earthquake, vertical distribution of acceleration increase ratio calculated by Armenian standard is expected as excessive value.

Calculation of vertical distribution of acceleration increase ratio shall be discussed in Detail Design Stage to design appropriate dam structure.

b) Evaluation of shearing strength of non-cohesive materials

There are some methods to evaluate shearing strength of non-cohesive materials. Major evaluation is shown in Table 6-5-6.20 and in Figure 6-5-6.14. Method No.2 and No.3 is based on the theory that non-cohesive material has no value of cohesion (c) and No.2 method, considering internal friction as only a factor of shearing strength, is selected in this Survey. Internal friction angle is estimated by field survey, not by laboratory tests. On the other hand, some value of c is applied for non cohesion materials in the design conducted during Soviet era.

Shearing strength under low lateral pressure σ is quite different depending on the methods especially. Therefore in Detail Design Stage, laboratory test and appropriate evaluation of shearing strength targeting non-cohesive materials shall be conducted.

Table 6-5-6.20 Major Evaluation Methods of Shearing Strength of Non-Cohesive Material

No.	Outline of Method	Formula
1	c and φ are calculated utilizing the results of laboratory test.	$\tau=c+\sigma \tan\varphi$
2	c and φ are calculated utilizing the results of laboratory test but only φ is applied as shearing strength factor.	$\tau=\sigma \tan\varphi$
3	τ is shown by an exponential function and parameter A and b is calculated utilizing the results of laboratory test	$\tau=A\,\sigma^b$

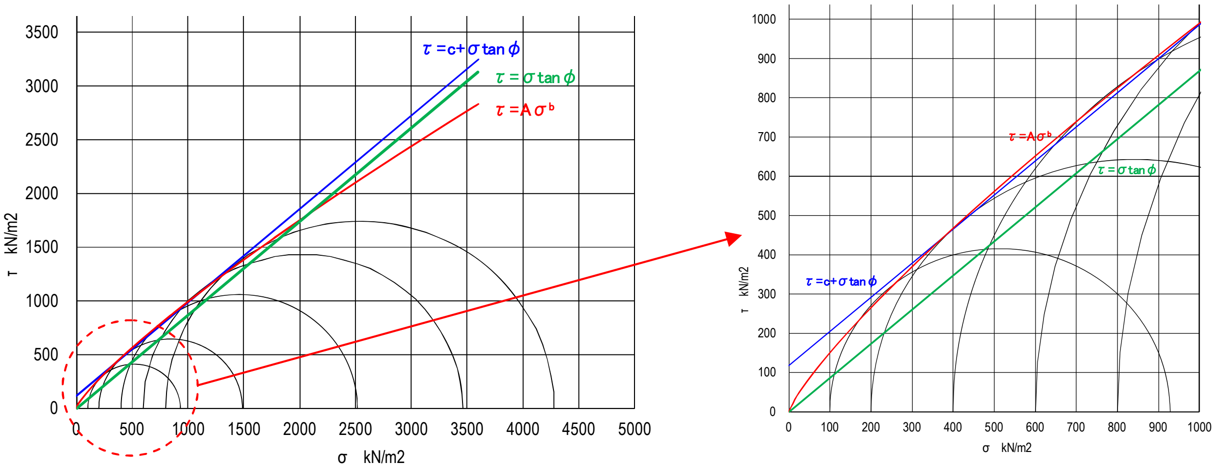


Figure 6-5-6.14 Major Evaluation Method of Shearing Strength of Non-Cohesive Material

6-5-7 Basic Design of Related Facilities (Emergency Discharge Structure)

(1) Specific condition for Yeghvard reservoir

As already described in "4-3-8 Situations Related to the Safety of Facilities," emergency discharge volume shall be examined taking into consideration Yeghvard reservoir's own situations shown as below.

a) Main emergency situation is damage on the reservoir by earthquake,
b) Destination of discharging is Kasakh Rrver,
c) Water is discharged through pile line
d) Facilities along Kasakh river will suffer from flood damage in case huge volume of water is discharged from Yeghvard reservoir and,
e) For Nor Yerznka village, water level shall be lowered as fast as possible (emergency discharge volume shall be as much as possible) to mitigate risk of dam collapse and damage in case dam collapse.

Taking into account these conditions, here sets two (2) kinds of emergency situations caused by earthquake shown as below and discharge volume is set for each condition.

Low Emergency (Low possibility of dam collapse)

[Conditions]
- Some observed parameters indicate mild abnormal tendency such as increasing of leakage volume or decreasing of water pressure regardless of the fluctuation of water level.

[Measure]
- Discharge water with its volume less than flow capacity of Kasakh River

High Emergency (High possibility of dam collapse)

[Conditions]
- Some observed parameters indicate serious abnormal tendency such as sudden increasing of leakage volume or sudden decreasing of water pressure regardless of the fluctuation of water level.
- Some deformations which indicate sliding failure of dam body such as faulting at upper area or swelling at lower area.

[Measures]
- Make alarming to Nor Yerznka village to evacuate to high land area to mitigate damage by flood caused by dam collapse
- Make alarming along Kasakh river to notice to the people to evacuate far from the river to mitigate damage by flood caused by emergency discharging
- Discharge water with maximum volume (*Along Kasakh river will be flooded.)
- Discharge water through outlet No.1 as well (*Beneficiary area covered by Arzni Branch canal will be flooded.)

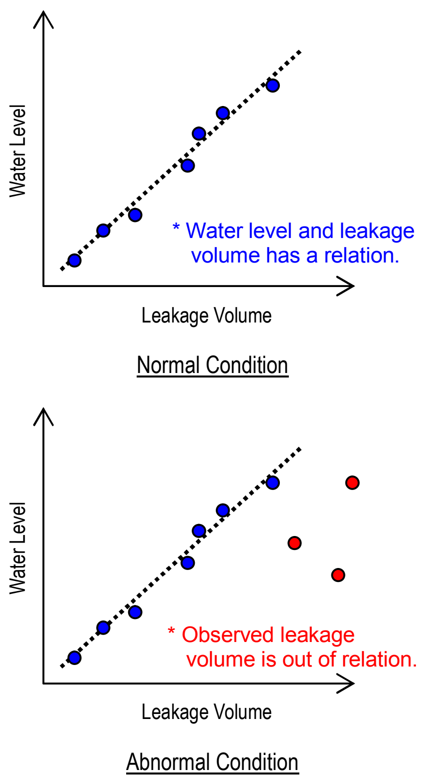


Figure 6-5-7.1 A sample of Abnormal Trend (Leakage Volume)

(2) Discharge volume under Low Emergency Condition

1) Flow capacity of Kasakh River

Interview survey targeting main facilities along Kasakh river is conducted to grasp the historical flood damage. Figure 6-5-7.2 shows the location of target facilities and Table 6-5-7.1 shows the summary of survey results.

According to the results, it is judged that maximum discharge volume at Ashtarak observation station (almost same as discharge destination point) which does not cause flood along Kasakh river is 13.7m^3/s and this value is selected as flow capacity of Kasakh river.

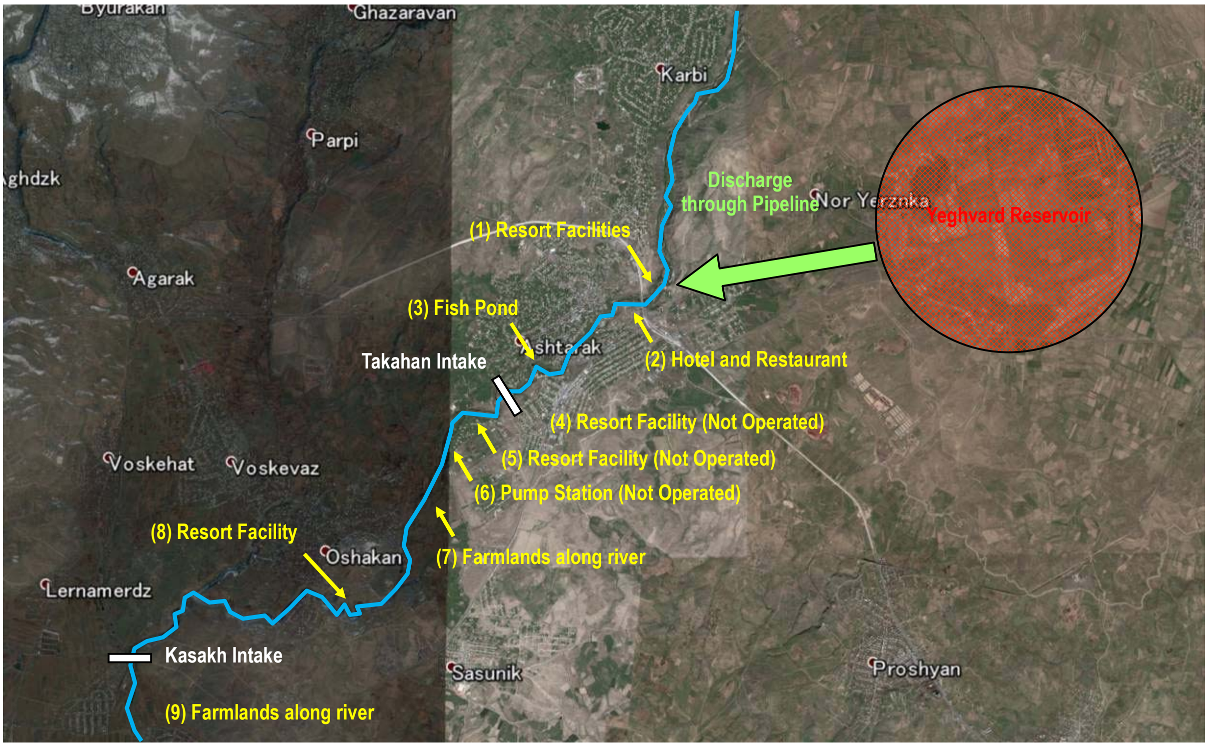


Figure 6-5-7.2 Location of Main Facilities along Kasakh River

Table 6-5-7.1 Summary of the Interview Survey Results

No.	Target Facility		Results on Interview Survey	Flow Capacity
1	Resort Facilities		- Facilities are located at relatively higher area to mitigate flood damage. - River bank protection was constructed by the fund of the owner. **The height of protection is higher than the water level which occurs as in previous years.** - Even if facilities suffer damage from flood, no compensation is provided from government. Rehabilitation of the facilities is done by the fund of the owner.	13.7m^3/s (Minimum Flood volume in the record)
2	Hotel and Restaurant		- Facilities are constructed 4 or 5 years ago. - **After construction, no flood damage has happened.** *Interview results of guard man because the owner was not there	42.2m^3/s (Minimum Flood volume for last 4 years)

No.	Target Facility		Results on Interview Survey	Flow Capacity
3	Fish Pond		- Flood damage has not happened on the facilities after interviewee started work as a guard man (started year is not sure). - Flood damage has not happened on intake facilities as well. *Interview results of guard man because the owner was not there	$13.7m^3/s$ (Minimum Flood volume in the record)
4	Resort Facility		- Facilities are not operated and could not contact the owner. Same as the other facilities, it is assumed that **this facility has safety against flood which occurs as in previous year.**	$13.7m^3/s$ (Minimum Flood volume in the record)
5	Resort Facility		- Facilities are not operated and could not contact the owner. Same as the other facilities, it is assumed that **this facility has safety against flood which occurs as in previous year.**	$13.7m^3/s$ (Minimum Flood volume in the record)
6	Pump Station		- Facilities are not operated and could not contact the owner. Same as the other facilities, it is assumed that **this facility has safety against flood which occurs as in previous year.**	$13.7m^3/s$ (Minimum Flood volume in the record)
7	Farmlands along the river		- There is a channel at the upstream edge of the farmlands to divert a part of flood so that **serious damage has not happened by the flood occurs as in previous year.** - The area damaged by flood occurs as in previous years are limited to the area just besides the river.	$13.7m^3/s$ (Minimum Flood volume in the record)
8	Resort Facility		- Facilities are located at relatively higher area. Therefore same as the other facilities, it is assumed that **this facility has safety against flood which occurs as in previous year.** *Interview survey was not conducted.	$13.7m^3/s$ (Minimum Flood volume in the record)
9	Farmlands along the river		- There is river bank protection along the river so that **flood damage has not happened for last 14 or 15 years.**	$130m^3/s$ (Minimum Flood volume for last 15 years.)

*"Flood volume" means the annual maximum volume at Ashtarak observation station.

2) Discharge volume from Yeghvard reservoir

At the destination of discharging, there is a flow from upstream. Therefore the gap of volume between flow capacity of Kasakh river and flow from upstream can be discharge volume from Yeghvard reservoir Q_Y as shown in the Figure 6-5-7.3.

Q_Y varies according to the season and water level of Yeghvard reservoir varies as well. Therefore, Q_Y by water level is defined as shown in the Figure 6-5-7.4. Discharge facility is designed with capacity to discharge at least this volume at each water level.

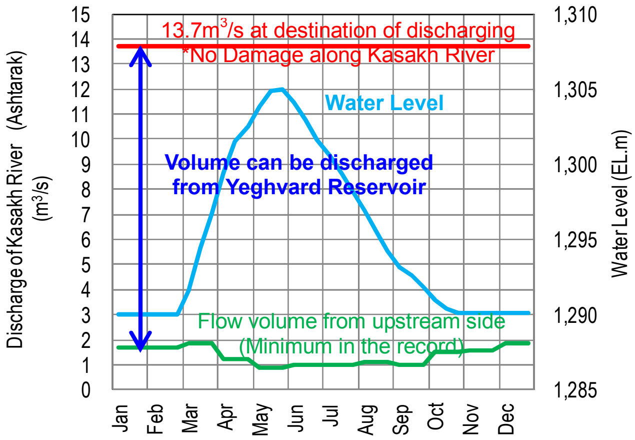


Figure 6-5-7.3 Discharge volume from Yeghvard Reservoir

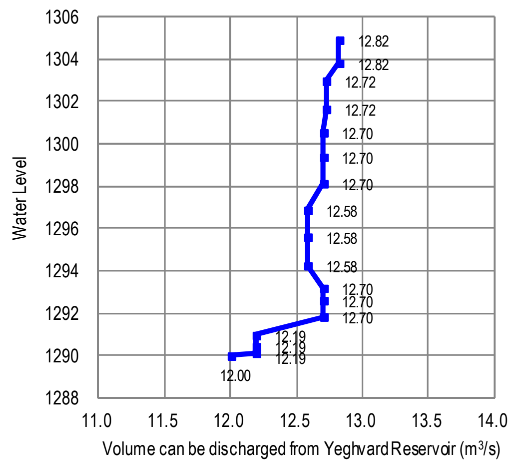


Figure 6-5-7.4 Design Condition of Emergency Discharge Facility

(3) Discharge volume under high emergency condition

In case of High Emergency Condition, discharge control valve is fully opened and maximum volume of water is discharged to lower water level as soon as possible.

The maximum discharge volume of each water level is shown in the Figure 6-5-7.5. In this case, discharge volume is more than the flow capacity of Kasakh river and areas along the river is flooded.

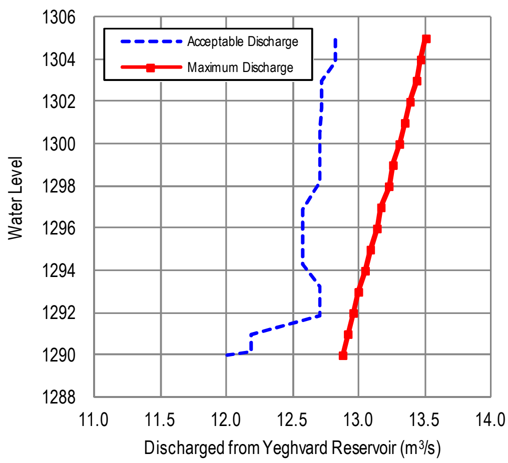


Figure 6-5-7.5 Discharge volume under High Emergency Condition (=Maximum Discharge Volume)

Also it takes about 80 days to lower the water level from FWL to LWL. There are some standards prescribing days to empty the reservoir or velocity of water level lowering in the other countries' standard, such as i) Empty reservoir within 10 days or ii) Lower water level with velocity 1m/day (Kaps applies this prescribing). For a dam constructed crossing river, downstream side will not be flooded even if stored water is discharged according to the prescribing above because downstream side is developed with safety capacity against flood and this flood volume is bigger than emergency discharge amount. However as already described, Yeghvard reservoir is constructed closing plane area by two (2) dam bodies and destination of discharging is Kasakh river, condition is quite different from general dams. If prescribing for emergency discharge above is applied to Yeghvard reservoir, scale of discharge facility becomes very huge and development of downstream side of discharging destination is needed. In this case huge amount of construction cost is required. Therefore it is judged that just to apply other countries' prescribing is not appropriate for Yeghvard reservoir and original regulation shall be defined.

(4) Operation plan of emergency discharge

Table 6-5-7.2 shows tentative operation procedure of emergency discharge after earthquake and concerning matters in each step. The detail examination, especially threshold to conduct each activity, will be conducted during Detail Design stage.

Table 6-5-7.2 Operation Procedure of Emergency Discharging (Tentative)

		Activity	Concerning matter	Necessary items
1	Happening of Earthquake	- Confirm scale of earthquake by data observed by devises	-	- Seismometer
2	Data collection	- Collect observed data, such as leakage volume and water pressure - Check if observed data shows abnormal trend	-	- Leakage measurement structure - Water pressure devise - Water level gauge - System to compile observed data
3	Patrolling	- Patrol and check the condition of structures *Dam body* Sliding failure, deformation, cracks *Concrete structure* Cracks *Boundary of concrete and soil structure* Leakage *Gate* Deformation of gate, shaft, door stop	-	- Patrol vehicle(s)
4-1	Data is normal and there are no strange event	- To continue normal operation	-	-
4-2	Trend of data or condition of structure is judged as Low emergency condition	- Discharge to Kasakh River with volume less than flow capacity of Kasakh River	Total discharge volume from Yeghvard Reservoir and upstream shall be less than flow capacity of Kasakh River (13.7m^3)	- Data transfer system from Ashtarak station to operation system
4-3	Trend of data or condition of structure is judged as High emergency condition	1) Alarming to Nor Yerznka village to evacuate to higher area (to mitigate damage in case dam collapse) 2) Alarming to the area along Kasakh River to evacuate far from river (to mitigate damage by flood caused by emergency discharge) 3) Discharge to Kasakh River with volume less than flow capacity of Kasakh River 4) Confirm all the persons in and around Khasakh River evacuate 5) Open discharge control valve fully and discharge maximum volume	3) Total discharge volume from Yeghvard Reservoir and upstream shall be less than flow capacity of Kasakh River (13.7m^3)	- Data transfer system from Ashtarak station to operation system - Alarming system to Nor Yerznka village - Alarming system to the area along Kasakh River - Evacuation plan for Nor Yerznka village and areas along Kasakh River

6-5-8 Safety Facilities of the Dams and the Reservoir

(1) Safety control of the dams and the reservoir

(a) Safety control of the dams

In fill-type dam’s case, it is the standard to monitor the leakage quantity from the dam body, deformation of the dam body and the seepage condition in the dam body as the safety control of the dam.

As for the leakage from the dam body, a measurement system composed of a channel and a weir shall be installed at the toe of the downstream of dam body slope to Dam No.1 and Dam No.2.

As for the deformation monitoring, a deformation survey network and survey facilities for checking deformation after an earthquake shall be introduced to the whole area of downstream/upstream slope and the dam crest to Dam No.1 and Dam No.2.

As for the seepage condition, it is usual to install the wells for observing seepage water table to grasp the seepage condition as ”a seepage line”. But in case of Dam No.1 and Dam No.2, the impervious zone is provided with as an inclined core zone the width of which is narrow and most of which lay under water beneath the upstream slope so that it is difficult to install the observation wells and get the accurate data regarding the seepage water table. Considering such points, pore pressure gauges shall be installed in place of the observation wells.

(b) Safety control of the reservoir

For the reservoir of which slopes are completely covered by anti-infiltration works, backpressure behind the anti-infiltration works is crucial to keep the storage function of the reservoir in normal because the excess backpressure can easily destroy the anti-infiltration works due to its light weight. Some tens of pore pressure gauges shall be installed to check and monitor the backpressure condition and to grasp the occurrence of abnormal conditions.

(2) Monitoring of leakage from the reservoir

Monitoring of the leakage by the observation wells shall be done in the long span of time and area. The disadvantage of pore pressure gauges is mortality due to the measurement system being maintained by electricity. Monitoring system by wells shall not function as the keen system to catch the abnormal condition quickly but function effectively to catch the change of condition in the long span of time. Several to about ten observation wells about 30m deep shall be installed around the reservoir except for the four (4) deep observation wells already installed in this preliminary survey stage.

(3) Safety facilities for the maintenance works and the visitors

(a) Safety facilities to the maintenance work

The maintenance or surveillance work shall be executed by vehicles. To avoid vehicles dropping accidentally from the dam crest, a row of safety barricade by placed rocks shall be installed at along the edge of the dam crest.

(b) Safety facilities for the visitors

Parks for the recreation activities of inhabitants shall be constructed, where circumstances and facilities for the visitors to enjoy the water safely shall be prepared and arranged.