# Annex-6 Cua Can Impounding Reservoir

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### *1.2.1 Legal Documents concerning the Preliminary Design Preparation*

1. Vietnamese Standards TCVN 4118 – 85: Foundations of Irrigation Works – Design standards.
2. Branch Standards 14TCN 157 – 2005: Design standards for compacted soil dam.
3. Design Regulations QP.TL.C-8-76: Regulations on Hydraulic calculation of Spillway .
4. TCXD VN 285 : 2002 Irrigation Works–Main Processes of Design.
5. Regulations of Loads and Forces on Irrigation works( by Waves and Boats ), 14TCN 28-85-QPTL.C.1.78
6. 14TCN 130-2002: Design Guidelines for sea dyke
7. TCXDVN 375:2006 –Design forBuilding againt Earquake (acceleration=0.004, under grade V of MSK-64).
8. Norms applied for calculating the designed hydrological characters, 14TCN 10-85-QPTL.C.6.77

### *1.2.2 Main Design cricteria*

*<u>1 Work Level</u>*

*a) Level of Head Works:*

Level of Works: under TCXDVN 285:2002

-A resevor with storage volume of 10,54 millions $m^3$ will be in arange from (1 – 20) millions $m^3$ : the level IV

- Head Works: Soil dam with a dam height of H = 8.5 m, in arrange of 8-15m,belong to group B, the glevel of dam is: the level IV

- Water Supply flow: 52.500 $m^3$/day equivalent to 0.61 $m^3/s < 2$ $m^3/s$, the grade of Work is the IV grade.

Therefore, the level of Head Works are: **Level IV**

*<u>2. Design Standards</u>*

*a) Standards applied for designing the Head Works:* According to TCXDVN 285:2002, with the work level IV. Designed Norms as follows:

- Designed Flood frequency: 1,5%

- Checked flood Frequency: 0,5%

- Flood Frequency of the construction flow diversion: 10%

- Permible safety Factor, Ensuring Factor, load combination factor, load deflection factor applied for designing main facilities of the head works under TCXDVN 285:2002 complied with the work level IV.

*b) Standard applied for Water Supply*
- Frequency of ensuring water supply for domestic & other use:  95%

## 1.3 General Introduction

### *1.3.1 Location of the Project*

The Cua can Reservoir will be built on an area next Cai Stream, in Cua Duong Commune, Phu Quoc Island district, Kien Giang province. The Reservoir locates on the North of Phu Quoc Island, 18 km far from Duong Dong town the following travel road. A center of the Reservoir locates around at latitude $10^{o}$18’58’’ North and Longitude $103^{o}$57’41’’ East.

### *1.3.2 Summaryze  main technical standards*

**a. Functions of Work :**

- Supply raw water for the Water Treatment Plant with acapacity of Phase 1 : 21,000 $m^3$/day and Phase 2: 52,500 $m^3$/day.
- Discharge water to maintain the environmental flow and improve the area’s environmenta  in dry season.

**b. Work Level and Designed Frequency:**

*Standards applied for designing the Head Works:* According to TCXDVN 285:2002, with the IV level work. Designed Norms as follows:

- Designed Flood frequency: 1,5%

- Checked flood Frequency: 0,5%

- Frequency of flooding diversion for construction: 10%

- Safety factor, ensured factor, load combination factor, load deviation factor for designing main items of headworks followed the TCXDVN 285:2002 corresponding to the works level  IV.

*Standard applied for Water Supply*
- Frequency of ensuring water supply for domestic & other use:  95%

**c. Technology and Technical Alternatives**

1. **The water storage reservoir** : is a annual regulation reservoir
2. **Work Functions :** Supplying raw water for the Water Treatment Plant with acapacity of Phase 1 : 21,000 $m^3$/day and Phase 2: 52,500 $m^3$/day. Discharging water to maintain the environmental flow and improve the area’s environmenta  in dry season.
3. **Location of the reservoir plan:** Cua can reservoir plan has been located based on the orientation of keeping the original state of Cua Can river; the reservoir will be built on area next to the river and water will be taken to the reservoir through intake culvert and pump. The reservoir is expanded to the Southwest around 60ha in compared to the Phu Quoc Island Adjusted Master Plan.
4. **Capacity of the Reservoir shows in below table:**

| No | The Reservoir Capacity | Unit | Phase 1 | Phase 2 |
|---|---|---|---|---|

| | | | | |
|---|---|---|---|---|
| 1 | The reservoir bottom elvation | m | 5,0 | 5,00 |
| 2 | Dead water level | m | 5,60 | 5,90 |
| 3 | Normal water level | m | 7,20 | 10,50 |
| 4 | Dead Volume: $W_c$ | $10^6$ $m^3$ | 1,13 | 1,69 |
| 5 | Total volume: $W_{tp}$ | $10^6$ $m^3$ | 4,15 | 10,47 |
| 6 | Effective volume: $W_{hi}$ | $10^6$ $m^3$ | 3,02 | 8,77 |

5. **Anti-seepage for the reservoir:** Because the reservoir bottom elevation is designed at +5.0m that most reservoir bottom area lies on the clay soil layer of CL or sandy-clay soil layer of SC1 which are strongly capable to prevent the water infiltrated through the reservoir bottom and the anti-seepage method applied for the reservoir bottom layer does not need to consider. However, It is proposed to estimate a quantity of clay soil for spreading a layer of 1m thick above the 20% of the total reservoir bottom area to prevent seepage for the reservoir.

6. **Earth dam Surrounding the Reservoir:** the Soil Berm is made of the soil excavated from the reservoir bed. The berm has its crest elevation of +12.0m and mounted on with a breakwater wall reaching to an elevation of 13.2m.. The soil berm crest gets 5m wide, reinforced with 20 cm thick aggregate stone layer and asphaltic penetration of 5.5kg/m2. The upstream side of embankment slope will be reinforced with concrete slab of 18cm thick and underneath with aggregate stone layer of 10cm thick and a fabric layer.

7. **Spillway:** The spillway is made of reinforced concrete, free-style overflow, practical section with dimension of 3x10m, overflow crest‘s elevation of +4.10m, designed discharge quantity of $Q_{1.5\%}$= 1083 $m^3$/s.

8. **Deep Oulet Culvert:** this is a reinforced concrete box culvert equipped with flat gate valve operated by electrical motor; the box culvert has a dimension of of 2x(2.5x2.5)m and invert level of +1.5mm

9. **Water intake Pumping station:** the pumps used are submersible motor pumps, the pumping house is made of concreted and the total capcity of the pumping station is 4.97 $m^3$/s. In the pump sump, there is a diversed sewer for maintenance of environmental flow. The parameters for pump station in the following table:

| No. | Specifications | Values | | |
|---|---|---|---|---|
| 1 | Pump | PL 7065/735 | LL 3300 LT | NL 3300 LT |
| 2 | No. of Pumps | 3 | 2 | 1 |
| 3 | Pump Head | 8 | 8 | 8 |
| 4 | Pumping Flow (l/s) | 1375 | 340 | 165 |
| 5 | The Largest pump performance (%) | 79.5 | 77.8 | 78.6 |
| 6 | Code of wheel work | 640 | 801 | 821 |

| | | | | |
|---|---|---|---|---|
| 7 | Motor power | 140 | 37 | 27 |
| 8 | Effluent Shaft power (Kw) | 136.9 | 37 | 21 |
| 9 | Inlet Shaft power (Kw) | 147.1 | 35 | 18 |
| 10 | Rpm (rounds/mimute) | 980 | 725 | 725 |
| 11 | 3-phase voltage | 380 | 380 | 380 |

10. **Progress of executing the works :** proposed 4 years.
11. **Total construction cost :** 398,027,630,000 VN dong.
12. **Investment phases :** all items will be synchronizedly invested in phase1.

**WORKS SPECIFICATIONS**

| No | Main facilities's specifications | Unit | Propsed Value |
|---|---|---|---|
| ***1*** | ***The Work levels*** | | *Level IV* |
| ***2*** | ***Specifics of Catchemnt and annual flow*** | | |
| | - Catchment area (Flv) | km² | 84 |
| | - Average rainfall of many years (Xo) | mm | 2833 |
| | - Average height of run-off for many years (Yo) | mm | 478 |
| | - Mô-đun of run-off (Mo) | l/s/km² | 52.38 |
| | - Average flow for many years (Qo) | m³/s | 4.16 |
| | - Total quantity (Wo) | million s.m³ | 130.24 |
| ***3*** | ***The phase 1 reservoir*** | | |
| | - a Frequency to ensure supplying water | | P = 95% |
| | - The Reservoir bottom elevation | m | 5,0 |
| | - Dead water level | m | 5,60 |
| | - Normal water level | m | 7,20 |
| | - Dead Volume $W_c$ | tr.m³ | 1,13 |
| | - Total volume $W_{tb}$ | mill.m³ | 4,15 |
| | - Effective volume $W_{hi}$ | mill.m³ | 3.02 |
| ***4*** | ***The phase 2 reservoir*** | | |
| | - a Frequency to ensure supplying water | | P = 95% |
| | - The Reservoir bottom elevation | m | 5,00 |
| | - Dead water level | m | 5,90 |
| | - Normal water level | m | 10,50 |
| | - Dead Volume $W_c$ | mill.m³ | 1,69 |

| No | Main facilities's specifications | Unit | Propsed Value |
|---|---|---|---|
| | - Total volume $W_{tb}$ | mill.m³ | 10,47 |
| | - Effective volume $W_{hi}$ | mill.m³ | 8.77 |
| ***5*** | ***Dam*** | | |
| | - Dam alignment | | Surround the reservoir |
| | - Dam type | | Soil berm |
| | - Dam crest elvation | m | 12.0 |
| | - Elevation of the breakwater wall'crest | m | 13.2 |
| | - The reservoir bottom elevation | m | 5.0 |
| | - Dam length | m | 5489 |
| | - Dam width | m | 5.0 |
| | - the largest height of the Dam | m | 7.00 |
| | - Reinforcing the upstream slope of the Dam (the reservoir side) | | Concrete slab |
| | - Reinforcing the downstream slope of the Dam (the river side) | | Grass planting |
| | - Anti-infiltration for Dam and Reservoir | | Clay soil layer of 1m thick |
| ***6*** | ***Spillway*** | | |
| | - Location | | On the river bed |
| | - Designed flood flow P = 1,5% | m³/s | 1083 |
| | - Checked flood flow P = 0,5% | m³/s | 1285 |
| | - Mode of flood discharge | | Free overflow |
| | - Flooding water level | m | +4.10 |
| | - Overflow crest elevation | m | +4.10 |
| | - overflow width | m | 3x10 |
| ***7*** | ***Deep effluent culvert*** | | |
| | - Location | | In the river bed |
| | - Designed effluent flow | m³/s | 88.0 |
| | - Mode of flood relieve | | Deep effluent culvert with gate |
| | - Invert level of the Culvert | m | +1.50 |
| | - Number of gate | | 2 |
| | - Dimension of gate | B x H | 2.5x2.5m |
| ***8*** | ***Water intake Pumping Station*** | | |

| No | Main facilities's specifications | Unit | Propsed Value |
|---|---|---|---|
| | - House structure | | Reinforced Concrete |
| | - Desinged Flow | $m^3/s$ | 4.97 |
| | - Pump Head | m | 7 |
| | - No. of Pumps | set | 6 |
| | - Elevation of discharged point | m | +11.0 |
| | - Elevation of suction pit bottom | m | +2.00 |
| | - Pumps | | Submersible motor pumps |

# CHAPTER 2 NATURAL CONDITIONS

## 2.1. TOPOGRAPHICAL CONDITIONS OF THE PROJECT AREA

### *2.1.1 Geographical Location*

The Cua can Reservoir will be built on an area next Cai Stream, in Cua Duong Commune, Phu Quoc Island district, Kien Giang province. The Reservoir locates on the North of Phu Quoc Island, 18 km far from Duong Dong town the following travel road. A center of the Reservoir locates around at latitude $10^{o}$18’58’’ North and Longitude $103^{o}$57’41’’ East.

### *2.1.2 Topography*

*a) State of available documents*

The topography of Phu Quoc Island area has been quite detailed measured. the topographical aerial photographs have the following scale:

- Aerial photographical maps of the project area with a scale of 1/2,000
- Aerial photographical maps of the project area with a scale of 1/5,000
- Aerial photographical maps of the reservoir bed area with a scale of 1/2.000

In addition, in December, 2011, the cross-sections of Cai stream have been measured at five locations to document for the basis design in 12/2011.

*b) Topography of Catchment Basin and the Reservoir Bed Area*

A terrain of the catchment basin slopes gradually from West to East with the highest peak elevation of about +552.2 m. the Catchment Basin shaped hilly terrain with steep slopes of relatively large, averaging about 20%. The 33km long Cai-stream flows from West to East, near the center of the Catchment basin and divides it into North and South sub-catchments. the stream has an averge slope of about 0.4%.

Cua Can reservoir bed area got altitude from about + 4.0m to + 8.0m, is a relatively flat area, about 8.0 km far from the river estuary to the sea. This type of terrain is entirely unfavorable for constructing the reservoir surrounded with soil dam or the reservoir must be constructd by digging soil to make below ground surface reservoir instead of damming up to to create aboved ground reservoir.

## 2.2. Geological

### *2.2.1 Earthquake Activities.*

According to seismic zoning map of TCXDVN 375:2006 - Design buildings against earthquakes, the Cua Can region has predicted in the earthquake level of 5, the acceleration a = 0.004, below the level of 5 of MSK-64 scale. This is a small earthquake level, causing no damage or destroy the building.

### *2.2.2 Evaluation on water storing capability of the Cua can Reservoir.*

According to the cross-sections prepared in the geological surveyed documents of the reservoir area and a general survey of the reservoir bed area, it shows that the distribution of natural soil layers of CL (low plasticity clay) with a small permeability and SC1 (light clayed sand) having midle permeability, these two soil layers are common distributed and can keep the water in the reservoir; they naturally have a thickness from 2m to over 5m. Distribution beneath the aboved two soil layers of CL and SC1 is SC soil layer (sandy clay – lightly grained sand) of rather high thickness and a high permeability. Therefore, It's possible to conclude that keeping the water in the reservoir depends on the retaining of the original soil layers of CL and SC1 with a thickness sufficient to ensure the water proof layer.

### *2.2.3 Geotechnical conditions at The Reservoir Area*

#### Geotechnical condition at the soil berm site

- SC1 soil layer : light- medium clayey Sand, dark grey, medium dense. Thickness is small of 0.7m and only met at the D4
- SC2 soil layer: Medium- light clayey sand, whitish grey - yellowish - reddish brown, soft, medium dense. Thickness varied from 2.8m to > 5.0m
- SC soil Layer: Clayey fine sand, whitish grey, medium dense, saturated.The layer is only met at the D1 with thickness unknown. At the D4 is the thin lense (0.8m), and is not met at D3.
- SW soil Layer: Fine sand, whitish grey, saturated, medium dense, well graded. This layer is met only at D4 ( thin lense 0.8m thickness)
- CL soil Layer: Clay - Very Clayey sand, yellowish grey - reddish brown - whitish grey, stiff - very stiff . Thickness is >3.0m

Table 2.1: **Result of physico -mechanical propeties of soil layers at soil berm**

| Properties | Layer SC2 | Layer SC | Layer CL |
|---|---|---|---|
| Grain size distribution | | | |
| Clay % | 15 | 6 | 35 |
| Silt % | 11 | 5 | 16 |
| Sand % | 72 | 89 | 49 |
| Gravel % | 2 | - | - |
| Atterberg limits | | | |
| Liquid limit $W_L$ % | 27 | - | 46 |
| Plastic limit$W_P$ % | 14 | - | 26 |
| Plasticity index $I_P$ | 13 | - | 20 |
| Liquidity index B | 0.31 | - | 0.02 |
| Natural moisture content W % | 18.0 | 16.7 | 26.5 |
| Wet density $\gamma_W$ T/ $m^3$ | 2.03 | 2.05 | 1.96 |
| Dry density $\gamma_d$ T/ $m^3$ | 1.72 | 1.75 | 1.55 |
| Specific gravity $\Delta$ | 2.67 | 2.66 | 2.68 |
| Porosity n % | 35.7 | 34.1 | 42.1 |

| | | | |
|---|---|---|---|
| Void ratio ε | 0.556 | 0.517 | 0.727 |
| Saturation degree G % | 86.5 | 85.6 | 97.6 |
| Direct shear test | | | |
| Cohesion C $kg/cm^2$ | 0.19 | 0.06 | 0.31 |
| Internal friction angle φ ( $^0$ ) | $22^0 33'$ | $29^0 21'$ | $16^0 33'$ |
| Module elasticity $E_0$ $Kg/cm^2$ | | | |
| $P_{0.0-0.5}$ | 14.18 | 51.37 | 19.77 |
| $P_{0.5-1.0}$ | 21.21 | 60.04 | 23.02 |
| $P_{1.0-2.0}$ | 34.75 | 89.46 | 28.94 |
| $P_{2.0-4.0}$ | 59.39 | 209.29 | 37.74 |
| Permeability coefficient K cm/s | $9.2x10^{-5}$ | $7.9x10^{-3}$ | $1.4x10^{-5}$ |

Table 2.2: **Result of triaxial compression test type UU (soil berm)**

| Lab. N$^O$ | Borehole | Depth of sample (m) | Soil layer | Result | |
|---|---|---|---|---|---|
| | | | | $C_{u-u}$ ($kg/cm^2$) | $\varphi_{u-u}$ ( $^0$ ) |
| 17 | D1 | 2.4-3.0 | CL | 0.51 | $12^0 56'$ |
| 21 | D3 | 4.4-4.8 | SC2 | 0.26 | $11^0 23'$ |
| 24 | D4 | 1.2-1.6 | CL | 0.39 | $10^0 28'$ |

**Geotechnical condition at the regulator gate**

- Layer SC:Clayey fine sand, whitish grey - yellowish grey, medium dense,saturated, well graded Thickness varied from 2.0 - > 3.0m
- Layer SW: Fine sand with few small gravel of quartz, whitish grey, light yellow, saturated, medium dense. Thickness varied 5.0 - >8.0m
- Layer CL: Clay, whitish grey, brownish grey , stiff - very stiff . This layer is met only at T2, thickness 1.0 - 4.0m
- Layer CG.: Residual of bed rock - Sandstone. Clayey Sand reddish brown, yellowish - whitish grey, very stiff to hard. Thickness varied from 4.5m to 6.0m
- Layer CL1:Residual of bed rock - Sandstone. Clayey Sand reddish brown, yellowish , very stiff to stiff. Thickness is unknown..

Table 2.3: **Result of physico mechanical properties of soil layers at regulator gate**

| Parameter | Layer SC | Layer SW | Layer CL | Layer CG | Layer CL1 |
|---|---|---|---|---|---|
| Grain size distribution | | | | | |
| Clay % | 8 | 1 | 38 | 8 | 27 |
| Silt % | 9 | 5 | 21 | 7 | 18 |
| Sand % | 82 | 89 | 41 | 67 | 55 |
| Gravel % | 1 | 5 | - | 18 | - |
| Atterberg limits | | | | | |

| | | | | | |
|---|---|---|---|---|---|
| Liquid limit $W_L$ % | - | - | 50 | - | 41 |
| Plastic limit$W_P$ % | - | - | 26 | - | 22 |
| Plasticity index $I_P$ | - | - | 24 | - | 19 |
| Liquidity index B | - | - | -0.11 | - | 0.07 |
| Natural moisture content W % | 14.4 | 13.1 | 23.5 | 19.6 | 23.6 |
| Wet density $\gamma_W$ T/ $m^3$ | 2.00 | 1.98 | 1.99 | 2.05 | 2.32 |
| Dry density $\gamma_d$ T/ $m^3$ | 1.74 | 1.75 | 1.61 | 1.71 | 1.88 |
| Specific gravity $\Delta$ | 2.66 | 2.65 | 2.65 | 2.67 | 3.05 |
| Porosity n % | 34.4 | 34.1 | 39.2 | 35.8 | 38.4 |
| Void ratio $\varepsilon$ | 0.525 | 0.516 | 0.646 | 0.558 | 0.624 |
| Saturation degree G % | 72.8 | 67.0 | 96.3 | 93.4 | 94.8 |
| Direct shear test | | | | | |
| Cohesion C kg/$cm^2$ | 0.08 | 0.04 | 0.37 | 0.32 | 0.30 |
| Internal friction angle $\varphi$ ( $^0$ ) | $28^0 54^/$ | $30^0 34^/$ | $16^0 17^/$ | $15^0 00^/$ | $19^0 46^/$ |
| Module elasticity $E_0$ Kg/$cm^2$ | | | | | |
| $P_{0.0-0.5}$ | 53.00 | 51.39 | 20.49 | 19.73 | 20.06 |
| $P_{0.5-1.0}$ | 60.38 | 59.92 | 22.98 | 22.25 | 24.05 |
| $P_{1.0-2.0}$ | 100.03 | 99.17 | 29.29 | 28.20 | 32.56 |
| $P_{2.0-4.0}$ | 178.86 | 187.92 | 38.05 | 37.15 | 46.36 |
| Permeability coefficient K cm/s | - | - | $5.15x10^{-6}$ | - | $1.5x10^{-5}$ |

Table 2.4: **Result of triaxial compression test type UU ( Regulator gate)**

| Lab. $N^O$ | Borehole | Depth of sample (m) | Soil layer | Result | |
|---|---|---|---|---|---|
| | | | | $C_{u-u}$ (kg/cm$^2$) | $\varphi_{u-u}$ ( $^0$ ) |
| 33 | T2 | 2.8-3.2 | CL | 0.57 | $14^0 12^/$ |

**Geotechnical condition at the intake gate**

- Layer SC1:Clayey Sand, whitish grey - yellowish - reddish brown, medium dense. Thickness varied from 1.9m to 2.6m
- Layer SC2:Very Clayey Sand - Clayey Sand whitish grey - yellowish brown,stiff. Thickness varied from 4.2 - 4.4m
- Layer SC:Clayey fine sand, whitish grey - yellowish grey, medium dense,saturated, well graded. Thickness varied from 2.3 to > 4.0m
- Layer CL:Clay - Very Clayey sand yellowish grey - reddish brown - whitish grey, stiff - very stiff . Thickness varied from 1.6 - 4.7m.

Table 2.5: **Result of physico mechanical properties of soil layers at intake gate**

| Parameter | Layer SC1 | Layer SC2 | Layer SC | Layer CL |
|---|---|---|---|---|
| Grain size distribution | | | | |
| Clay % | 14 | 20 | 6 | 37 |
| Silt % | 18 | 16 | 5 | 17 |
| Sand % | 68 | 64 | 88 | 46 |
| Gravel % | - | - | 1 | - |
| Atterberg limits | | | | |
| Liquid limit $W_L$ % | 27 | 30 | - | 48 |
| Plastic limit $W_P$ % | 15 | 16 | - | 27 |
| Plasticity index $I_P$ | 12 | 14 | - | 21 |
| Liquidity index B | 0.22 | 0.08 | - | -0.04 |
| Natural moisture content W % | 17.6 | 17.5 | 15.9 | 25.9 |
| Wet density $\gamma_W$ T/ m$^3$ | 1.90 | 2.03 | 2.04 | 1.98 |
| Dry density $\gamma_d$ T/ m$^3$ | 1.62 | 1.73 | 1.76 | 1.57 |
| Specific gravity $\Delta$ | 2.62 | 2.65 | 2.66 | 2.68 |
| Porosity n % | 38.3 | 34.9 | 33.7 | 41.3 |
| Void ratio $\varepsilon$ | 0.622 | 0.536 | 0.507 | 0.705 |
| Saturation degree G % | 74.2 | 86.4 | 83.2 | 98.4 |
| Direct shear test | | | | |
| Cohesion C kg/cm$^2$ | 0.16 | 0.25 | 0.08 | 0.38 |
| Internal friction angle $\varphi$ ( $^0$ ) | $23^0 07^/$ | $18^0 30^/$ | $29^0 31^/$ | $17^0 38^/$ |
| Module elasticity $E_0$ Kg/cm$^2$ | | | | |

| | | | | |
|---|---|---|---|---|
| $P_{0.0-0.5}$ | 25.80 | 23.49 | 50.79 | 23.36 |
| $P_{0.5-1.0}$ | 29.76 | 30.03 | 62.13 | 27.89 |
| $P_{1.0-2.0}$ | 34.67 | 39.15 | 88.87 | 36.27 |
| $P_{2.0-4.0}$ | 44.46 | 53.69 | 129.49 | 47.54 |
| Permeability coefficient K cm/s | - | - | - | $7.0x10^{-6}$ |

**Geotechnical conditions at the Water Treatment Plant**

- Layer CL: Clay - Very Clayey sand yellowish grey - reddish brown - whitish grey, stiff - very stiff. Thickness is 1.9m
- Layer CH: Clay, dark grey - dark brown -grey, soft - very soft. Thickness is 2.7m, intercalated with some thin layers silty sand.
- Layer SC: Clayey fine sand with some peat or plant decayed, whitish grey - yellowish grey, medium dense,saturated. Thickness is > 4.1m

Table 2.6: **Result of physico mechanical parameters of soil layers at Water Treatment Plant**

| Parameter | Layer CH | Layer SC |
|---|---|---|
| Grain size distribution | | |
| Clay % | 43 | 5 |
| Silt % | 19 | 6 |
| Sand % | 38 | 89 |
| Gravel % | - | - |
| Atterberg limits | | |
| Liquid limit $W_L$ % | 44 | - |
| Plastic limit$W_P$ % | 23 | - |
| Plasticity index $I_P$ | 21 | - |
| Liquidity index B | 0.40 | - |
| Natural moisture content W % | 30.9 | 14.7 |
| Wet density $\gamma_W$ T/ $m^3$ | 1.78 | 2.03 |
| Dry density $\gamma_d$ T/ $m^3$ | 1.36 | 1.77 |
| Specific gravity $\Delta$ | 2.64 | 2.65 |
| Porosity n % | 48.6 | 33.2 |
| Void ratio $\varepsilon$ | 0.947 | 0.498 |
| Saturation degree G % | 86.1 | 77.9 |
| Direct shear test | | |
| Cohesion C kg/$cm^2$ | 0.20 | 0.07 |
| Internal friction angle $\varphi$ ( $^0$ ) | $10^0 25^/$ | $29^0 55^/$ |
| Module elasticity $E_0$ Kg/$cm^2$ | | |
| $P_{0.0-0.5}$ | 9.32 | 76.55 |

| | | |
|---|---|---|
| $P_{0.5-1.0}$ | 12.94 | 81.05 |
| $P_{1.0-2.0}$ | 20.46 | 118.28 |
| $P_{2.0-4.0}$ | 26.73 | 220.58 |
| Permeability coefficient K cm/s | - | $2.79x10^{-3}$ |

Table 2.7: **Result of triaxial compression test type UU (WTP)**

| Lab. $N^O$ | Borehole | Depth of sample (m) | Soil layer | Result | |
|---|---|---|---|---|---|
| | | | | $C_{u-u}$ (kg/cm$^2$) | $\varphi_{u-u}$ ( $^0$ ) |
| 53 | P1 | 3.4-3.8 | CH | 0.26 | $10^0 56^/$ |

**Required Material for the project construction**

- Layer CL: Clay - Very Clayey sand, yellowish grey - reddish brown - whitish grey, stiff - very stiff .
- Layer SC: Clayey fine sand, whitish, yellowish grey, medium dense. the SC layer beneath the CL soil layer is aqueous layer and a little part above the CL layer is non-aqueous layer

Table 2.10: **Result of Standard Proctor test of samples at borrow area**

| Lab. $N^o$ | Soil layer | Borehole | Depth of sample (m) | Grain size distribution | | | | Atterberg limits | | | Standard Proctor | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Clay <0.005 | Silt 0.005-0.05 | Sand 0.05 2.0 | Gra. >2.0 | $W_L$ % | $W_P$ % | $I_P$ | $\gamma_d^{max}$ T / m$^3$ | $W_{op}$ % |
| 65 | CL | VL1 | 1.4-4.0 | 32 | 15 | 51 | 2 | 39 | 20 | 19 | 1.649 | 16.4 |
| 66 | | VL2 | 0.0-2.6 | 19 | 15 | 65 | 1 | 31 | 17 | 14 | 1.754 | 14.5 |
| 67 | | VL3 | 0.0-3.6 | 32 | 13 | 52 | 3 | 42 | 23 | 19 | 1.725 | 17.2 |
| 68 | | VL4 | 0.0-1.6 | 32 | 12 | 54 | 2 | 35 | 18 | 17 | 1.715 | 16.2 |
| 69 | | VL5 | 0.0-4.0 | 46 | 18 | 36 | - | 48 | 25 | 23 | 1.671 | 18.5 |

The samples is remoulded with the condition: $\gamma_d^{remoud} = 0.98\ \gamma_d^{max}$

And the result of direct shear and consolidation , permeability test as follows:

Table 2. 11: **Result of physico mechanical properties remoulded soil of borrow area**

| Parameter | Layer CL |
|---|---|
| Grain size distribution | |
| Clay % | 32 |
| Silt % | 14 |
| Sand % | 52 |
| Gravel % | 2 |

| | |
|---|---|
| Atterberg limits | |
| Liquid limit $W_L$ % | 39 |
| Plastic limit$W_P$ % | 21 |
| Plasticity index $I_P$ | 18 |
| Liquidity index B | |
| Moisture content W % | 18.9 |
| Wet density $\gamma_W$ T/ $m^3$ | 1.98 |
| Dry density $\gamma_d$ T/ $m^3$ | 1.67 |
| Specific gravity $\Delta$ | 2.70 |
| Porosity n % | 38.1 |
| Void ratio $\varepsilon$ | 0.616 |
| Saturation degree G % | 82.7 |
| Direct shear test | |
| Cohesion C kg/$cm^2$ | 0.39 |
| Internal friction angle $\varphi$ ( $^0$ ) | $15^0 37^/$ |
| Module elasticity $E_0$ Kg/$cm^2$ | |
| $P_{0.0-0.5}$ | 21.29 |
| $P_{0.5-1.0}$ | 26.02 |
| $P_{1.0-2.0}$ | 33.89 |
| $P_{2.0-4.0}$ | 42.06 |
| Permeability coefficient K cm/s | $3.5x10^{-6}$ |

Table 2.12: **Result of triaxial compression test type UU (Remoulded sample borrow area)**

| Lab. N$^O$ | Borehole | Depth of sample (m) | Soil layer | Result | |
|---|---|---|---|---|---|
| | | | | $C_{u-u}$ (kg/$cm^2$) | $\varphi_{u-u}$ ( $^0$ ) |
| 65 | VL1 | 1.4-4.0 | CL | 0.52 | $10^0 50^/$ |
| 67 | VL3 | 0.0-3.6 | CL | 0.45 | $10^0 22^/$ |

- Quantity of CL layer suitable for use calculated as follows:

Area of borrow soil of 2.5ha, the average explotation thickness of1.9m

Exploitation quantity = 25000$m^2$ x 1.9m = 47.500$m^3$

Caution: the exploitation thickness remain 1.0m for the cover layer of bottom reservoir for impervious layer ( covered the SC, not to allow appear the SC layer).

The CL layer expands large in the reservoir and can exploit

## 2.3. Meteo-hydrology.

### *2.3.1 Meteo-Hydological Gauging Stations and State of available Documents.*

In Suoi Cai stream catchment basin, there is no gauging-station. The nearest and unique one is the Hydro-meteorological gauging-station located in Duong Dong town, which covers about 15km streams catchment basin and are used to identify the hydro-meteorological characteristics for the works.

Table 2.8: Network of Rainfall and Meteo gauging-stations

| No | Name of station | location | Type of station | Observation time |
|---|---|---|---|---|
| 1 | Phu Quoc | $103^o58$'Đ; $10^o15$'B | Sea Hydro-meteology | 1957 – 2011 |

*(Source : the Hydro-meteorological gauging-station of the South Vietnam)*

In the Project Basin there is no hydro guaging- station and there are some hydro guaging-stations Inland at similar latitudes, due to no availble hydrological data in Phu Quoc, It is able to use documents of these gauging-stations for reference in the calculation. Details of the stations are as follows:

Table 2.9: The neigbouring Hydro Gauging-Stations

| No | Name of Station | Location | River | $F_{lv}$ (km$^2$) | Observation time |
|---|---|---|---|---|---|
| 1 | Dai Nga | $107^o$ 52'Đ; $11^o32$'B | Đa Reng | 361 | 1974 till now |
| 2 | Can Dang | $105^o60$'Đ; $11^o32$'31"B | Ben Da | 617 | 1977 till now |

*(Source : the Hydro-meteorological gauging-station of the Western Higlands, the Hydro-meteorological gauging-station of the South Vietnam)*

*Assessment on documents' quality* : quality of monitoring documents of these hydro gauging- stations to ensure reliability, documentation for many years, updated to the latest time (2010), may well use for reference or as similar stations in the calculation determining features for the works construction design.

### 2.3.2 Meteorological Characteristics

Cua Can stream of Phu Quoc District, Kien Giang province brings features of a tropical monsoon climate. Hot, humid and rainy, annual climate differentiated into two distinct seasons, rainy and dry seasons.

A Dry season from November to April of next year is dominated by dry winds and the rainfall in this season only obtains 10% - 15% of annual rainfall. Weather in dry season is nearly all hot, especially the last month of the dry season.
The rainy season from May to October, affecting primarily monsoon, in additions, the project area is located between the West Sea area which carries more moisture to cause rains, rainfall of this season accounted for 85 to 90% of annual rainfall. This is the period of heavy rains due to the activities of the tropical cenverging strips, tropical low pressure areas and the influence of the West Sea storms.

The project catchment basin is one of the protected forest areas in Phu Quoc island; thereforce, this forest area has good coverage. Characteristics of the Cua Can stream catchment basin are flowing through the quite diversified terrain, Cua Can stream starts at a region with a heigth from 450 to 500 m. Water from upstream normally flows between high cliffs in highly steep run-off that flood quickly formed.

***i. Temperature***

Table 2.10: Air temperature

| Specifics | month | | | | | | | | | | | | Year round |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | năm |
| Tmax | 25,6 | 26,2 | 27,4 | 28,2 | 28,5 | 27,8 | 27,5 | 27,5 | 27,1 | 26,3 | 26,4 | 25,9 | 27,0 |
| Tmin | 27,5 | 27,9 | 29,1 | 29,7 | 30,3 | 28,6 | 28,4 | 27,9 | 27,8 | 27,4 | 27,2 | 27,2 | 30,3 |
| Ttb | 23,9 | 24,7 | 26,2 | 26,8 | 27,8 | 27,1 | 26,8 | 27,0 | 26,2 | 25,1 | 25,4 | 24,0 | 23,9 |

*(Source : the Hydro-meteorological gauging-station of the South Vietnam).*

***ii. Air Humidity***

Table 2.11: Air humidity

| Specifics | month | | | | | | | | | | | | Year round |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | năm |
| Ubq | 76,5 | 77,1 | 77,6 | 80,2 | 83,0 | 85,1 | 85,6 | 86,1 | 86,6 | 86,2 | 80,3 | 74,4 | 81,6 |
| Umax | 80,0 | 81,0 | 81,0 | 84,0 | 84,0 | 87,0 | 88,0 | 88,0 | 89,0 | 89,0 | 84,0 | 83,0 | 89,0 |
| Umin | 74,0 | 74,0 | 74,0 | 76,0 | 78,0 | 82,0 | 83,0 | 85,0 | 85,0 | 82,0 | 74,0 | 67,0 | 67,0 |

*(Source : the Hydro-meteorological gauging-station of the South Vietnam).*

***iii. Wind on the ground***

\+ There are 2 windy seasons : Winter and summer windy seasons; the prevalent windy direction is west and east directions, Average velocity of wind is 4.0m/s.

Table 2.12: main specifics of wind

*(Unit: m/s)*

| Specifics | month | | | | | | | | | | | | year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| $V_{bq}$ | 3,7 | 3,4 | 3,3 | 2,9 | 3,3 | 5,1 | 4,9 | 5,0 | 4,8 | 2,9 | 3,6 | 4,1 | 4,0 |
| Directio | E | E | E | E | WE | W | W | W | W | W | NW | E | WE |
| $V_{max}$ | 16 | 17 | 14 | 26 | 24 | 30 | 34 | 34 | 21 | 40 | 40 | 21 | 16 |

*(Source : the Hydro-meteorological gauging-station of the South Vietnam).*

Table 2.13: Maximum velocity of wind under direction

*(Unit: m/s)*

| Frequency | Wind direction | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | N | NE | E | SE | S | SW | W | NW |
| 2% | 20,5 | 26,7 | 32,0 | 15,5 | 17,5 | 28,5 | 30,3 | 43,6 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 4% | 18,3 | 24,3 | 25,6 | 14,1 | 16,3 | 26,1 | 28,4 | 37,6 |
| 10% | 15,1 | 21,0 | 17,8 | 12,2 | 14,4 | 22,5 | 25,5 | 29,5 |
| 25% | 11,6 | 17,6 | 12,2 | 9,92 | 12,3 | 18,7 | 22,3 | 22,0 |
| 50% | 8,5 | 14,6 | 9,98 | 7,78 | 10,3 | 15,3 | 19,5 | 16,7 |

*(Source : the Hydro-meteorological gauging-station of the South Vietnam).*

Table 2.14: Maximum velocity of wind exclusive of wind direction

*(Unit: m/s)*

| Frequency | 2% | 4% | 10% | 25% | 50% |
|---|---|---|---|---|---|
| $V_{maxp}$ exclusive wind direction | 47,4 | 41,7 | 33,9 | 26,4 | 20,9 |

*(Source : the Hydro-meteorological gauging-station of the South Vietnam).*

***iv. Evaporation***

In the whole year round, the strongest evaporation happens in dry season months, especially in December and January that are not the months having highest temperature of year but low moisture. In the rainy season, evaporation is reduced markedly, especially in September and October; in these months it gets heavy rains and high humidity.

Table 2.15: Monthly Average Evaporation during year round at Phu Quoc Monitor Station

*(Unit:mm)*

| Character | Months | | | | | | | | | | | | *year* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| Average Epiche | 118,3 | 98,3 | 106,7 | 95,3 | 91,7 | 85,4 | 80,3 | 81,8 | 73,1 | 71,7 | 112,4 | 145,6 | 1160,6 |

*(Source :The South Vietnam Meteo-Hydrological Station).*

+ From the existing available data, the calculation of evaporation for Cua Can Catchment Basin are follows:

- Average annual loss of the Catchment Basindetermines by the water balance equation as follows:

$$Z_{oLV} = X_{oLV} - Y_{oLV}$$

*Of which :*

$X_{oLV}$ :as the average perticipitation of a Catchment Basin, from available data, it can identify $Xo_{LV}$ = 2883mm
$Y_{oLV}$ :is the average flow, $Yo_{LV}$ = 1652mm
$Z_{oLV}$ :is the average annual water loss of the Catchment Basin

$$Z_{oLV} = X_{oLV} - Y_{oLV} = 2883mm – 1652mm = 1231mm$$

Observation documents at the gauging-stations show that it gets only evaporation data measured by the “Piche tube”method; so we must use the relation documents between evaporation data measured by “GGI-3000 barrel”method and by “Piche tube” method to apply for Cua Can Catcment Basin calculation.

A quantity of evaporation measured by the ground “GGI-3000 barrel” method is usually equal to 2 times of quantity of evaporation measured by the “Piche tube” method.

ZA = 2.0 x 1231 = 2462 mm

According to documents of the Gauge Station on the Southeast Viet Nam, a quantity of evaporation measured by the water-surface “GGI-3000 barrel” method is equal to 0.75 of a quantity of evaporation measured by the “Piche tube” method. And hen the quatity of evaporation measured by the water-surface ‘GGI-3000 barrel” method at Phu Quoc island as follows:

- $Z_{GGI-3000(water)}$ = 0,75 x 2462 = 1847mm

Water-surfave vapor distribution of Cua Can took similar as Piche tube’s evaporation distribution at Phu Quoc station showed in the following table.

Table 2.21: Distribution of water-surface vapor in a year

*(Etb : mm)*

| Character | Month | | | | | | | | | | | | year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| Etb | 188,3 | 156,4 | 169,8 | 151,7 | 145,9 | 135,9 | 127,8 | 130,2 | 116,3 | 114,1 | 178,9 | 231,7 | 1847 |

***v. The Water Loss from Reservoir Evaporation***

If a reservoir has been built, a part of catchment basin that the reservoir covers will make a loss of water through evaporation increasing more than evaporation of the original catchment basin without reservoir. So a reservoir evaporation loss ( DZo) is a difference between water loss of the water-suface evaporation (Znc) and the flow’s evaporation in the basin($Z_{oLV}$).

Water loss increasing with a reservoir built is :

$\Delta Zo = Z_{nc} - Z_{oLV}$

$Z_{oLV}$ = 1222mm ( as calculated above)

From aboved values, supposed to put them to the above equations and ΔZo will be determined as follows:

$\Delta Zo = Zwater - Z_{oLV} = 1847 - 1231 = 616$ mm

Distribution of the reservoir evaporation loss shows in below table

Table 2.22: Distribution water loss from reservoir evaporation

| Character | Month | | | | | | | | | | | | year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| ΔZ(mm) | 62,8 | 52,2 | 56,6 | 50,6 | 48,7 | 45,3 | 42,6 | 43,4 | 38,8 | 38,1 | 59,7 | 77,3 | 616 |

***vi. Number of Sunny Hours.***

Total yearly average sunny hours is 2678.

The months having the high average number of sunny hours are from November to May next year, the average number of sunny hours in May is the maximum one with 262 hours. The months having average number of sunny hours are from the June to September, in which August is the month having the lowest average number of sunny hours with 183 hours.

Table 2.23: Numbers of Annual Sunny hours

*(Unit : h)*

| Character | Month | | | | | | | | | | | | *year* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| $N_{bq}$ | 262 | 227 | 270 | 259 | 225 | 195 | 187 | 183 | 191 | 201 | 236 | 243 | 2678 |
| $N_{max}$ | 307 | 269 | 350 | 336 | 338 | 333 | 316 | 344 | 309 | 310 | 300 | 313 | 3731 |
| $N_{min}$ | 198 | 95 | 208 | 153 | 152 | 120 | 93 | 65 | 99 | 124 | 165 | 177 | 1910 |

*(Source :the South Viet nam Meteo-hydrological gauging station)*

*Note :*

$N_{bq}$ : The average number of sunshine

$N_{max}$ : The maximum number of sunshine

$N_{min}$ : The minimum number of sunshine

### *2.2.3 Rain and Rainfall calculation:*

#### *i. Determining Annual Rainfall Standards of the Catchment Basin*

The annual rainfall standards of Cua Can catchment basin determined same as annual rainfall standards of Phu Quoc is 2883mm/year

#### *ii. Distribution of annual designed rainfall*

With such aboved rainfall data and the statistical parameters (Cv, Cs) determined from the annual rainfall data chain in the document of Phu Quoc rain measuring station, It can calculate the designed rainfall of the Cua Can basin such as the following table:

Table 2.16: Annual Designed rainfall

| Item | Xo | Cv | Cs | Frequency P% | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 20% | 50% | 75% | 80% | 85% | 90% |
| Railfall X (mm) | 3077 | 0,16 | 0,247 | 3264 | 2863 | 2562 | 2490 | 2407 | 2305 |

Table 2.25: Distribution of Designed rainfall

*X (mm)*

| P (%) | Month | | | | | | | | | | | | Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| 50 | 37,5 | 25,8 | 79,1 | 170,2 | 249,1 | 345,7 | 453,1 | 470,2 | 467,3 | 352,5 | 152,5 | 59,9 | 2863 |
| 75 | 33,5 | 23,1 | 70,8 | 152,3 | 222,9 | 309,3 | 405,5 | 420,8 | 418,2 | 315,4 | 136,5 | 53,6 | 2562 |
| 80 | 32,6 | 22,5 | 68,8 | 148 | 216,6 | 300,7 | 394,1 | 409 | 406,5 | 306,6 | 132,6 | 52,1 | 2490 |

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 85 | 31,5 | 21,7 | 66,5 | 143,1 | 209,4 | 290,6 | 381 | 395,3 | 392,9 | 296,3 | 128,2 | 50,4 | 2407 |
| 90 | 30,2 | 20,8 | 63,7 | 137 | 200,5 | 278,3 | 364,8 | 378,6 | 376,3 | 283,8 | 122,8 | 48,2 | 2305 |

### *iii. Rain Causing Flood in The Rainy Season*

Every year, from August to November, It usually appears heavy rain, lasting 3 to 5 days or more causing flood in the basin.

For just the average basin-type as Cua Can catchment basin, flood peak flow primarly depends on the largest quantity of rainfall per day. That the rainfall of the whole rain and the distribution of rainfall regulated by the catchment will create a total quantity of flooding water and flood performance line. Therefore, determinations of a rainfall and a pattern of short duration rains are needed for calculating the flood in the basin.

Table 2.17: the highest rainfall causing flood in the rainy season

| Sation | Xo (mm) | Cv | Cs/Cv | X0,2% (mm) | X0,5% (mm) | X1,0% (mm) | X1,5% (mm) | X5,0% (mm) | X10% (mm) |
|---|---|---|---|---|---|---|---|---|---|
| Phu Quoc | 149,7 | 0,56 | 1,96 | 505,3 | 450,9 | 409,0 | 394,8 | 307,9 | 261 |

### *iv. Rain Causing Flood in The  Dry Season*

During the months of transitional period between the rainy to dry season, or from dry to wet season, rains occur but not as great as rains in the wet season; However, sometimes It has some faily big rains caused medium and small floods which are affected the construction work and often called as the works excecuting floods. These of short duration rains last 1 to 2 days and the maximum rainfall will dertermine flood peak flow.

Statistical calculations on the largest daily rainfall in a dry season and transition months (from January to August) give the following results:

Table 2.27: the highest rainfall causing flood in the Dry season

| No | Period | Cv | Cs | Max. Daily Rainfall (mm) | |
|---|---|---|---|---|---|
| | | | | (P = 5%) | (P = 10%) |
| 1 | The whole season | 0,59 | 1,55 | 137,8 | 120,0 |

## *2.3.4 The Annual Water Current*

### *i. General Characters.*

Cua Can stream catchment basin has  roots shape with the main river flowing from East to West direction. The basin is narrow at the upstream, gradually expanding downstream. Development trends of the basin roughly balanced on both sides of the stream slopes. Basin has the high elevations decreased from East to West. The Catchment divided boundary crossing the moutainous peaks with altitude from 450m to 500m.

Considering the entire stream basin getting hills characteristics, slopes of the basin and river are relatively large. Covering layer is major with grey soil and scrub trees, good water permeability, low capacity of water retention that flows in dry season and rainy season get the big difference. In the Drought season, flows are usually small and because the basin shape is convenient for the formation of flood flows so flooding flows are often very large.

The hydrological researchs for Cua Can streams have very limited done, along the stream (to the works position) does not have any hydro gauging-station. Near the basin there is the Phu Quoc sea-hydrometeorological gauging-station. Such flow documents used for studying Cua can stream are limited, so the calculation of the Cua Can stream flows must refer to the other similar gauging-stations. The situation of such available materials is very difficult to determine the required flow specific characteristics. In addition to maximize the available measured data, it should analyze and use data of neigbor basins and on the whole region and to give reasonable results, ensure the design requirements.

### *ii. The Annual Flow Standards*

As It is a closed basin, the annual flows in Cua can streams are completedly borned from rainfall in the catchment basin. Distribution of rainfall during the year and the regulation of the basin will determine the type of flow process in streams. Annual flow is typically slower than in rainy season; the flood season is same in rainy season and exhausted season is in the dry season.

Determining the standard flow of Cua Can stream basin is based on Dai Nga Flow Measurement Station inland with a basin of 361km ².

Table 2.18: Monhtly average flow

*($Q$ : $m^3/s$)*

| Sation | Month | | | | | | | | | | | | year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| Dai Nga | 4.50 | 2.88 | 2.88 | 5.63 | 9.65 | 18.83 | 28.41 | 45.98 | 41.36 | 39.20 | 18.93 | 8.65 | 18.91 |

### *ii. Annual Flow Standard of Dai Nga Gauging-station*

From the availble measured data of Dai Nga Hydrological Stations, It can determine the standard flow as follows:

Table 2.19: Annual standard flow at Dai Nga Hydrological Station

| No | Name of Station | River name | Location | Catchment area (km²) | Qo (m³/s) | Mo (l/s – km²) |
|---|---|---|---|---|---|---|
| 1 | Dai Nga | Da Reng | Bao Loc – Lam Dong province | 361 | 18.91 | 52.38 |

### *iv. Annual Flow Standard of Cua Can Catchment Basin*

As in Cua Can catchment basin (counted up to the works alignment) there is no hydrological stations that the annual flow standard of the designed catchment basin can be calculated by determining the flow standard of Dai Nga station. Using the following formular:

$$Q_o = Q_{oÑNg} \frac{F_{CuaCaïn} X_{oCuaCaïn}}{F_{Ñai\,Nga} X_{0ÑaiNga}}$$

In which: $Q_o$, $Q_{oDNg}$: Annual flow standard at works alignment

In Dai Nga station ($Q_{oĐNg}$ = 18,91$m^3/s$)

$F_{Cua\ Cạn}$ , $F_{ĐaiNga}$ : Catchment area counted up to works alignment and counted

To Dai Nga Station ($F_{Cua\ Cạn}$=84,0km², $F_{Đai\ Nga}$ = 61km²)

$X_{oCua\ Cạn}$ , $X_{oĐai\ Nga}$ : Average rainfall of the Catchment counted up to works alignment and to Dai Nga station. ($X_{oCua\ Cạn}$ = 2883mm, $X_{oĐai\ Nga}$ = 2214mm)

Based on the string of recored rainfall data of 29 consecutive years ( from year 1979 to 2010) of Dai Nga gauging-station, it shall calculate the annual flow standard at the works alignment as follows:Qo = 5.41 ($m^3/s$)

Thus if the flow calculated from the above formula is big, a flow module of Cua Can basin will be larger of Dai Nga station flow module. On the Phu Quoc island, Duong Dong reservoir have been built and have its flow module (adopted as 53.9 l / s.km ²). From these analysis it is proposed to select the flow module of Cua Can Basin same as flow module of Dai Nga station, from which it calculate the flow of the technical options as follows:

**$Q_{o\ Cua\ Cạn}$ = 4,16 ($m^3/s$), water current coefficient $\alpha$ = 0.57**

### *v. Designed Annual Flow*

Calculating the coefficient Cv based on the formula $C_{vy} = C_{vx} (X_o/Y_o)$

- $C_{vy}$ : Coeff. of the flow fluctuation
- $C_{vx}$ : Coeff. of variation of rain
- $X_o$ : annual average rainfall
- $Y_o$ : Quantity of annual average current heigth.

$C_{vy}$ = 0.16 x ( 2883 / 1654) = 0.279

Selecting $C_{vy}$ = 0.28, Cs Taken according to standards Cs = 2Cv

+ From Qo, Cvy, Cs, a calculation of the designed annual flow are :+

Table 2.30: Designed Annual Flow

| No | Specific | P (%) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 25 | 50 | 75 | 85 | 90 | 95 |
| 1 | Qp ($m^3/s$) | 4.93 | 4.13 | 3.36 | 2.96 | 2.69 | 2.30 |
| | Wp ($10^6$m) | 155.47 | 130.24 | 105.96 | 93.35 | 84.83 | 72.53 |

### *vi. Distribution of Designed Annual Flow*

Based on a data sequence from the monthly average flow measused from Dai Nga gauging- station, using methods of similar basin applied for the designed work (defined above), using the Andreanop method to calculate the annual flow distribution. In that, year frequency of 25% taken from the distribution groups of years having abundant quantity of water, in year frequency of 50% taken from distribution group of years getting average quantity of water, in year frequency of 75% ÷ 100% obtained from distribution group in of exhausted years. The Calculation the flow distribution in the Cua Can basin is:

## * *Distribution of Annual Designed Flow calculated by Anđreanop Method*

From data of annual flow in calculation, using Andreanop method, the annual flow distribution as follows:

Table 2.31: Distribution of Annual Designed Flow calculated by Anđreanop Method

| P% | Qp | Month | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | ($m^3$/s) | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII |
| 1 | 7,01 | 1,388 | 0,897 | 0,680 | 2,152 | 4,073 | 9,190 | 8,601 | 19,986 | 15,829 | 12,092 | 6,267 | 2,965 |
| 5 | 6,13 | 1,214 | 0,785 | 0,595 | 1,882 | 3,562 | 8,036 | 7,522 | 17,477 | 13,842 | 10,574 | 5,480 | 2,593 |
| 10 | 5,67 | 1,123 | 0,726 | 0,550 | 1,741 | 3,294 | 7,433 | 6,957 | 16,165 | 12,803 | 9,781 | 5,069 | 2,398 |
| 25 | 4,93 | 0,976 | 0,631 | 0,478 | 1,514 | 2,864 | 6,463 | 6,049 | 14,055 | 11,132 | 8,504 | 4,407 | 2,085 |
| 50 | 4,13 | 0,681 | 0,525 | 0,376 | 0,909 | 2,110 | 3,259 | 5,530 | 13,588 | 7,504 | 9,326 | 4,229 | 1,520 |
| 75 | 3,36 | 0,699 | 0,373 | 0,296 | 0,528 | 1,045 | 2,379 | 4,976 | 6,347 | 8,145 | 10,030 | 3,978 | 1,522 |
| 90 | 2,69 | 0,560 | 0,299 | 0,237 | 0,422 | 0,837 | 1,905 | 3,984 | 5,081 | 6,521 | 8,030 | 3,185 | 1,219 |
| 95 | 2,30 | 0,478 | 0,255 | 0,202 | 0,361 | 0,715 | 1,628 | 3,406 | 4,345 | 5,575 | 6,866 | 2,723 | 1,042 |

### *2.3.5 Flooding Current*

As there is no flooding flow observation station in the stream basin counting up to proposed dam, therefore, Flooding flows are calculated from the maximum daily rainfall data.

#### *i. Designed Peak Flow of the Flood.*

The Cua Can stream basin has a medium size, the formation and factors influence to the flooding flows in the basin such as rainfall fed flooding, the state of buffer surface, topography of slopes and river beds presented in the previous items. The designed flood defined through relations of rain - flow by the normative formulas and with the stream catchment area <100 km ², It is able to use the ***Formula of Ultimate Rain Intensity ( the Rational method)*** specified in the regulations of hydrologic calculation of QP.TL . C - 6-77 as follows:

$$\mathbf{Q_{maxp} = Ap \times \varphi \times Hp \times F \times \delta}$$

In which :

Hp : Rainfall corresponding to Frequency

α : Coeeficint of flooding flow

F : Area of catchment basin

Ap : Module of peak flood depends on hydro-geomorphological characteristics of river

*$A_p$ defining based on the following characters :*

ϕd : Coefficient of slopes hydro-geomorphology

ϕs : Coefficient of river bed hydro-geomorphology

$$\phi_s = \frac{1000L}{m.J^{1/3}.F^{1/4}(\varphi.H_p)^{1/4}} \qquad \phi_d = \frac{(1000b_c)^{0.6}}{m_d.J_d^{0.3}.(\varphi.Hp)^{0.4}}$$

L : Length of the main river

m : Parameters of water concentration in river

md : Parameters of water current concentration on the slopes

J : Slope of the main river bed

Jd : slope of steep

bc : Average Length of the basin slope; bc = 1/(1,8ρ)

τd : Concentration time

ρ : Density of River network

Table 2.20: Results of Calculating $Q_{maxp}$ of the Catchement basin No. 1

| No. | Items | Frequency of Designed flood ( P%) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0,20 | 0,50 | 1,00 | 1,50 | 5,00 | 10,00 |
| 1 | Catchment area (F : km²) | 79,3 | 79,3 | 79,3 | 79,3 | 79,3 | 79,3 |
| 2 | Rainfall corresponding frequency (Hp : mm) | 505,3 | 450,9 | 409,0 | 394,8 | 307,9 | 261,0 |
| 3 | Coeff. of flooding flow ( j ) | 0,80 | 0,80 | 0,80 | 0,80 | 0,80 | 0,80 |
| 4 | Length of the main stream (L : km) | 23,79 | 23,79 | 23,79 | 23,79 | 23,79 | 23,79 |
| 5 | Length of the tributary stream (l : km) | 13,55 | 13,55 | 13,55 | 13,55 | 13,55 | 13,55 |
| 6 | Density of river network (r) | 0,471 | 0,471 | 0,471 | 0,471 | 0,471 | 0,471 |
| 7 | Parameters of flow concentration in river (m) | 9 | 9 | 9 | 9 | 9 | 9 |
| 8 | Parameters of flow concentration on the slopes ( md) | 0,25 | 0,25 | 0,25 | 0,25 | 0,25 | 0,25 |
| 9 | Slope of the main stream (J : ‰) | 3,8 | 3,8 | 3,8 | 3,8 | 3,8 | 3,8 |
| 10 | Slope of steep (Jd : ‰) | 211,2 | 211,2 | 211,2 | 211,2 | 211,2 | 211,2 |
| 11 | Average length of the basin slopes (bc) | 1,180 | 1,180 | 1,180 | 1,180 | 1,180 | 1,180 |
| 12 | Coeff. of the slopes hydro-geomorphology (fd) | 5,07 | 5,31 | 5,52 | 5,60 | 6,18 | 6,61 |
| 13 | Coeff. of the river hydro-geomorphology (fd) | 126,58 | 130,23 | 133,45 | 134,63 | 143,27 | 149,31 |
| 14 | Flow concentration time ( td ) | 27,180 | 28,74 | 30,13 | 30,65 | 34,72 | 37,97 |
| 16 | Module of flood peak (Ap) | 0,0463 | 0,0449 | 0,0436 | 0,0432 | 0,0401 | 0,0379 |
| | **Peak floding flow (m³/s)** | **1485** | **1285** | **1132** | **1083** | **784** | **628** |

Table 2.33: Calculation results $Q_{maxp}$ of Catchment basin No. 2

| No | Items | Frequency of Designed flood ( P%) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0,20 | 0,50 | 1,00 | 1,50 | 5,00 | 10,00 |
| 1 | Catchment area (F : km²) | 4,66 | 4,66 | 4,66 | 4,66 | 4,66 | 4,66 |
| 2 | Rainfall corresponding frequency (Hp : mm) | 505,3 | 450,9 | 409,0 | 394,8 | 307,9 | 261,0 |
| 3 | Coeff. of flooding flow ( j ) | 0,75 | 0,75 | 0,75 | 0,75 | 0,75 | 0,75 |
| 4 | Length of the main stream (L : km) | 3,34 | 3,34 | 3,34 | 3,34 | 3,34 | 3,34 |
| 5 | Length of the tributary stream (l : km) | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 | 0,00 |
| 6 | Density of river network (r) | 0,717 | 0,717 | 0,717 | 0,717 | 0,717 | 0,717 |
| 7 | Parameters of flow concentration in river (m) | 7 | 7 | 7 | 7 | 7 | 7 |

| No | Items | Frequency of Designed flood ( P%) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0,20 | 0,50 | 1,00 | 1,50 | 5,00 | 10,00 |
| 8 | Parameters of flow concentration on the slopes ( md) | 0,15 | 0,15 | 0,15 | 0,15 | 0,15 | 0,15 |
| 9 | Slope of the main stream (J : ‰) | 15,8 | 15,8 | 15,8 | 15,8 | 15,8 | 15,8 |
| 10 | Slope of steep (Jd : ‰) | 20 | 20 | 20 | 20 | 20 | 20 |
| 11 | Average length of the basin slopes (bc) | 0,775 | 0,775 | 0,775 | 0,775 | 0,775 | 0,775 |
| 12 | Coeff. of the slopes hydro-geomorphology (fd) | 13,67 | 14,31 | 14,88 | 15,09 | 16,67 | 17,80 |
| 13 | Coeff. of the river hydro-geomorphology (fd) | 29,33 | 30,18 | 30,92 | 31,20 | 33,20 | 34,60 |
| 14 | Flow concentration time ( td ) | 102,68 | 109,54 | 115,67 | 118,15 | 138,66 | 152,12 |
| 16 | Module of flood peak (Ap) | 0,0751 | 0,0718 | 0,0694 | 0,0684 | 0,0609 | 0,0561 |
| | **Peak flooding flow (m³/s)** | **132,6** | **113,1** | **99,2** | **94,4** | **65,5** | **51,2** |

Table 2.34: Summary of peak flooding Flows

| No. | Calculated Catchment | P (%) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0,20 | 0,50 | 1,00 | 1,50 | 5,00 | 10,00 |
| 1 | Catchment No. 1 | 1485 | 1285 | 1132 | 1083 | 784 | 628 |
| 2 | Catchment No. 2 | 132,6 | 113,1 | 99,2 | 94,4 | 65,5 | 51,2 |

### *ii. Designed Flood Process Curve and Total Flooding Flow*

Cua Can streams have no flooding measured data so it can use theoretical method to determine the flood process for the catchment basin as follows:

A line of Designed flooding process is established under GUDIC curve with the following type :

$$y = 10^{-\frac{a(1-x)^2}{x}}$$

In which :

$y = \frac{Qi}{Q'mp}$ : Is a flow y-ordinate of the designed flood process Qi, expressed as Decimal number compared with average flow of the maximum day Q'mp.

Q'mp = Qmp/KQn is the exchange coefficient depends on the catchment area, availble built in the table.

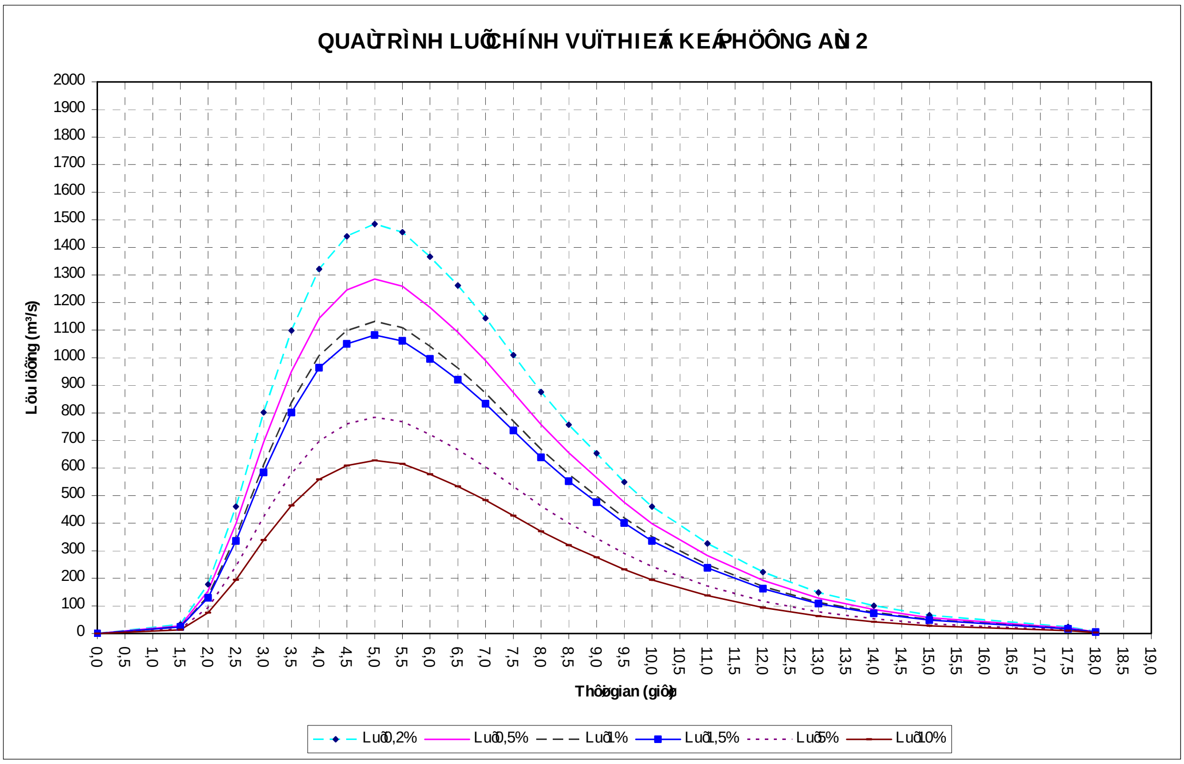
X=ti/tl : Is the x coordinate of the design flood distribution indicated by the decimal number compared to the conventional time tl

a : Parameter characterizes the shape of the flood process, depending on the coefficient of none - balance Ks of the flood or flood shape factor g(f)

From The relationship built between y and x with Ks, it can determine the designed flooding process

Table 2.35: a process of main designed floods of Catchment No. 1

| T(hour) | Flooding process Qi (m³/s) | | | | | |
|---|---|---|---|---|---|---|
| | 0,2% | 0,5% | 1,0% | 1,5% | 5,0% | 10,0% |
| 0,00 | 0 | 0 | 0 | 0 | 0 | 0 |
| 1,50 | 33 | 28 | 25 | 24 | 17 | 14 |
| 2,00 | 178 | 154 | 136 | 130 | 94 | 75 |
| 2,50 | 460 | 398 | 351 | 336 | 243 | 195 |
| 3,00 | 802 | 694 | 611 | 585 | 423 | 339 |
| 3,50 | 1099 | 951 | 838 | 801 | 580 | 465 |
| 4,00 | 1322 | 1144 | 1007 | 963 | 697 | 559 |
| 4,50 | 1440 | 1246 | 1098 | 1050 | 760 | 609 |
| **5,00** | **1485** | **1285** | **1132** | **1083** | **784** | **628** |
| 5,50 | 1455 | 1259 | 1109 | 1061 | 768 | 615 |
| 6,00 | 1366 | 1182 | 1041 | 996 | 721 | 578 |
| 6,50 | 1262 | 1092 | 962 | 920 | 666 | 534 |
| 7,00 | 1143 | 989 | 872 | 834 | 603 | 483 |
| 7,50 | 1010 | 874 | 770 | 736 | 533 | 427 |
| 8,00 | 876 | 758 | 668 | 639 | 462 | 370 |
| 8,50 | 757 | 655 | 577 | 552 | 400 | 320 |
| 9,00 | 653 | 565 | 498 | 476 | 345 | 276 |
| 9,50 | 549 | 475 | 419 | 401 | 290 | 232 |
| 10,00 | 460 | 398 | 351 | 336 | 243 | 195 |
| 11,00 | 327 | 283 | 249 | 238 | 172 | 138 |
| 12,00 | 223 | 193 | 170 | 162 | 118 | 94 |
| 13,00 | 149 | 129 | 113 | 108 | 78 | 63 |
| 14,00 | 101 | 87 | 77 | 74 | 53 | 43 |
| 15,00 | 67 | 58 | 51 | 49 | 35 | 28 |
| 17,50 | 24 | 21 | 18 | 17 | 13 | 10 |
| 18,00 | 7 | 6 | 6 | 5 | 4 | 3 |
| W ($10^6 m^3$) | 42,99 | 37,202 | 32,769 | 31,339 | 22,689 | 18,178 |




### *2.3.6The Biggest Flow in The Exhausted Season*

In the early and late dry season months it often appears heavy shower with high rain intensity and precipitation, causing significant floods that people often called as small early floods. The dry season is the season of irrigation works construction, so the identification of floods in the dry season is very important to have appropriate construction methods.

The project work is defined as level IV, according to TCXD VN 285: 2002 for which the frequency of flow and level of construction water diversion is 10%.

Construction peak flooding flows are determined from of the largest daily rainfall of the rain caused flooding in the basin in the dry months and transitional time between rainy season and dry season. Using the formula of the *Ultimate Rain Intensity* to calculate the construction flooding peak discharges (similar to calculate floodpeak discharges in main flooding season, with a rainfall on construction day from table 3.13 of Section 3.7.4 above). After computing the results of flows of construction water diversion show in the following table:

Table 2.21: Construction Flooding Peak Discharge

| No. | Items | Designed flooding frequency ( P%) | |
|---|---|---|---|
| | | 5,00 | 10,00 |
| 1 | Catchment area(F : km²) | 79,3 | 79,3 |
| 2 | Rainfall corresponding frequency (Hp : mm) | 137,8 | 120,0 |
| 3 | Coeff. of Flooding Current ( φ ) | 0,60 | 0,60 |
| 4 | Length of the main stream (L : km) | 23,79 | 23,79 |

| No. | Items | Designed flooding frequency ( P%) | |
|---|---|---|---|
| 5 | Length of the tributary stream (l : km) | 13,55 | 13,55 |
| 6 | Density of river network (r) | 0,471 | 0,471 |
| 7 | Parameters of flow concentration in river (m) | 9 | 9 |
| 8 | Parameters of flow concentration on the slopes ( md) | 0,25 | 0,25 |
| 9 | Slope of the main stream (J : ‰) | 3,8 | 3,8 |
| 10 | Slope of steep (Jd : ‰) | 211,2 | 211,2 |
| 11 | Average length of the basin slopes (bc) | 1,180 | 1,180 |
| 12 | Coeff. of the slopes hydro-geomorphology (fd) | 9,57 | 10,11 |
| 13 | Coeff. of the river hydro-geomorphology (fd) | 188,22 | 194,84 |
| 14 | Flow concentration time ( td ) | 61,39 | 66,13 |
| 16 | Module of flood peak (Ap) | 0,0296 | 0,0284 |
| | **Flooding Peak Discharge (m³/s)** | **194,2** | **162,2** |

Table 2.22: Construction Flooding Peak Flows according to different Period of Time

| No. | Flow according to period (P =10%) | | | | | |
|---|---|---|---|---|---|---|
| | XII-IV | XII-III | I - III | I - II | I | II |
| 1 | 121,0 | 104,1 | 87,7 | 62,6 | 33,8 | 52,2 |

### *2.3.7Mud and Sand Flow*

Cua Can stream basin has no survey for mud and sand flows. So to determine the flow of solid for the basin, it should refer to the actual measured data of solid flow on the rivers in the Southeast Viet Nam and with the empirical formula, combined the Sythetized -analysis method, expanding research for some other basins in the region; cover layer and geological structure.

***i.Dertermining the Quantity of Sand Holding***

*Applying formula :Po - lia - Kop:*

$$\rho o = 10^4 \varepsilon \sqrt{J}.K$$

In which

ρ : The quantity of suspended sand holding

e : Cavitation coeff.: this is a dense wooded area, less erosion, it should choose = 1,0

J : Slope of the river bed = 3,8 ‰

K : adjusted coef.: K = K1*K2*K3

K1 Adjusted coef. of the catchment basin’s section shape : selecting = 0,5

K2 Adjusted coef. of the catchment basin's vegetational cover: with forest both sides that selecting = 0,7

K3 Adjusted coef. of mineralogy: = 1,0

Choosing above appropriate parameters It can determine that $\rho$ = 216gram/m³.

*Reference from the approved neigbor projects:*

Duong Dong Reservoir constructed and in operation has proposed of $\rho$ = 120gram/m³.

From the aboved analysis results to choose a sand holding of the basin:

$\rho$ = 120gram/m³.

***ii. Total Annual Quantity of Sediments***

*Quantity of Suspended solid (SS)*

- Unit of SS transfer : Ro = ro.Qo (kg/s)
- Annual Quantity of SS : $G_{ll}$ = Ro.T (Ton/year)
- Annual Volume of SS: $W_{ll} = G/\gamma_1$ (m³)

  ($\gamma_1$ if density of SS)

*Amount of sediments settled on the reservoir bottom*

The amount of sediments settled on the reservoir bottom is empirically obatained 20% of SS quantity.

- Annual quantity of bottom settled down sediments: $G_{dđ}$ = 20% $G_{ll}$ (ton/year)
- Annual volume of bottom settled solid: $W_{dđáy} = G/\ \gamma_2$ (m³)

  ($\gamma_2$ is density of bottom settled solid $\gamma_2$ =1,5 ton/m³)

*Quantity of sediments fed from banks erosion*

Quantity of sediments fed from banks broken and quantity of vegetation Wsl is normally calculated based on the total quantity of suspended sediment and bottom sediment cell. Typically from 0.5 to 1.0%, in this case it is propose to choose K = 1.0%

***iii. Annual Total Quantity of Sediments to the Work's Alignment***

the total annual sediment load to the work's alignement of Wbc is a total amount of suspended sediment, bottom settled sediment and sediment obtanied from banks'lanslide. The calculated results show in the following table

Table 2.23: the total annual amount of sediment to the work's alignement

| Stt | Qo (m³/s) | Ro (kg/s) | Gll (tấn/năm) | Wll (m³) | Gdđ (tấn/năm) | Wdđ (m³) | Wsl (m³) | Wbc (m³) |
|---|---|---|---|---|---|---|---|---|
| 1 | 4,160 | 0,541 | 17.055 | 24.364 | 3.411 | 2.274 | 266 | 26.904 |

***iv. The Total amount of settled sediment at the work alignement***

+ Amount of settled sediment depenps on ratio

$$\beta_k = \frac{V_k}{W_0}$$

Wo is a flow of standard current

Vk is a capacity of water storage

if ik <0.15, the deposition ability between 40 and 50%, when 0.15 <ik <0.6, the ability of deposition ranges from 70 to 100%, while ik> 0.6, the ability of sedimentation deposition is 100% of the total sediment load to the work alignemnt. For Cua Can basin It is proposed to chosen the deposition capability of 100%.

### *2.3.8 Conclusions and Recommendations on the Catchement Basin Hydrology*

***i. Conclusions***

the Cua Can streams Basin has not been much studied on hydrology. However, in the neighbor areas there are many meteo-hydrological gauging-stations and meteorological monitoring sites with long consecutive recored and believable documents which can provide an amount of necessary information to reach requirements of analysis and calculations process for the meteo-hydological characteristics of the Cua Can basin.

In allowable conditions, with existing available document resources and applying the methods prescribed for the current calculations, the above studies have utilized the existing data, the reasonableness of analysis and safety by region, regional climate, hydrology, compared to basins with similar natural conditions. So these results at certain levels to ensure the accuracy and reasonable for use in the works design.

***ii. Recommendations***

During process of the project design, construction and operation It needs to implement the monitoring and survey measurement of fundamentally hydrological elements. At first, It immediate needs to take hydroloical survey (flow measurement), of annual mud flows and water quality at the work’s alignment. These factors are used as the actual basis for testing the parameters were calculated and adjusted if necessary.

The Cua Can reservoir has a small capacity, according to the calculations, amount of sediment deposition will be large, thereforce, it should plan to dredge the reservoir regularly to pevent it from filled up.

It is proposed to continue collecting meteo-hydrological documents, organizing investigations on historical flood and updating the relevant information to checked calculate the parameters of the project works in the next stage.

# CHAPTER 3 GENERAL PLAN AND WORK STRUCTURE

## 3.1 Arrangement of The general Reservoir Layout

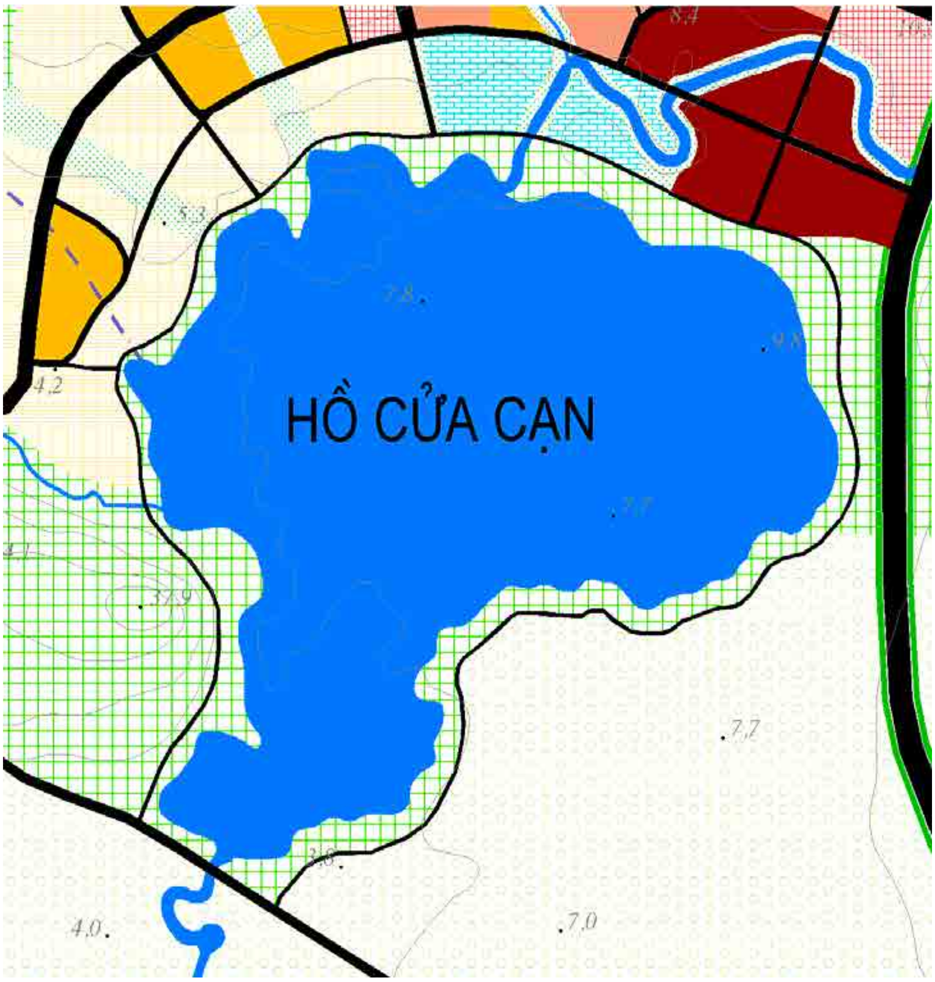
### *3.1.1 Layout of Reservoir according to the Master plan*

According to the Adjusted Master Plan to construct the Phu Quoc Island prepared by the South Urban- Rural Planning Sub-Institute in June, 2010, Cua Can reservoir fromed by constructing a dam on Cua Can river and the Cua Can Reservoir locates between Cua Can river and other small tributaries of Cua Can river.

Layout of Cua Can reservoirs in the Adjusted Mater Plan shows in Figure 3. 1.

Figure 3-1

Layout of Cua Can reservoirs in the Adjusted Mater Plan of year 2010.




Cua Can reservoir layout arranged as in the Phu Quoc Island Adjusted Master Plan has some demerits listed as follows:

- That the Cua Can stream is completely blocked to raise water level will lead to flooding at reservoir’s upstream because according to the aerial photograph maps, scale 1/5000, the low-lying areas along riversides and at about 2 km upstream of the Cua Can reservoir have the ground elevation of around +2.2 m; However, the topographical survey of cross-section of Cua Can river at the White Bridge position shows that elevation of the river bottom at this point is around 4.18m. As the river bottom elevation at the White Bridge is higher than the ground elevation (+2.2m) of upstream low-lying areas along Cua Can river, it can easily see that in the reservoir upstream areas there exists flooded areas even in the dry season. And according to a survey by local residents, it does not have any upstream flooded areas in the upstream side of the White Bridge and it gets abilities that the ground elevations in aerial photographical maps are not correct. For any reasons, based on terrain elevation of the Cua Can river bottom at the White bridge of 4.18m, It can confirm that to raise water level in the riverbed to +4.0m does not cause flooding in the Cua Can river upstream areas. With aboved reasons, the water level in Cua Can can only reach to +4.0m and to build the reservoir will require digging the soil area deeper and make a very big construction cost due to a huge quantity of excavated soil.

- According to the geological conditions of the reservoir area, a top layer of soil is clay with a thickness of about 6 meters and the next below soil layer is sand layer which gets water permeable. If the reservoir is deepened leading to the whole upper layer of clay to be peeled away it will make the reservoir lose water or require to build an anti-seepage layer which are too expensive.
- All sediment ( Sludge and sand) from upstream will flow through the reservoir, and almost all will be deposited in the reservoir as a velocity in the reservoir is almost equal to 0.

From the above disadvantages, It must be allocated the reservoir layout for more suitable than in the Master Plan.

### *3.1.2 Allocation of the Reservoir plan in this stage*

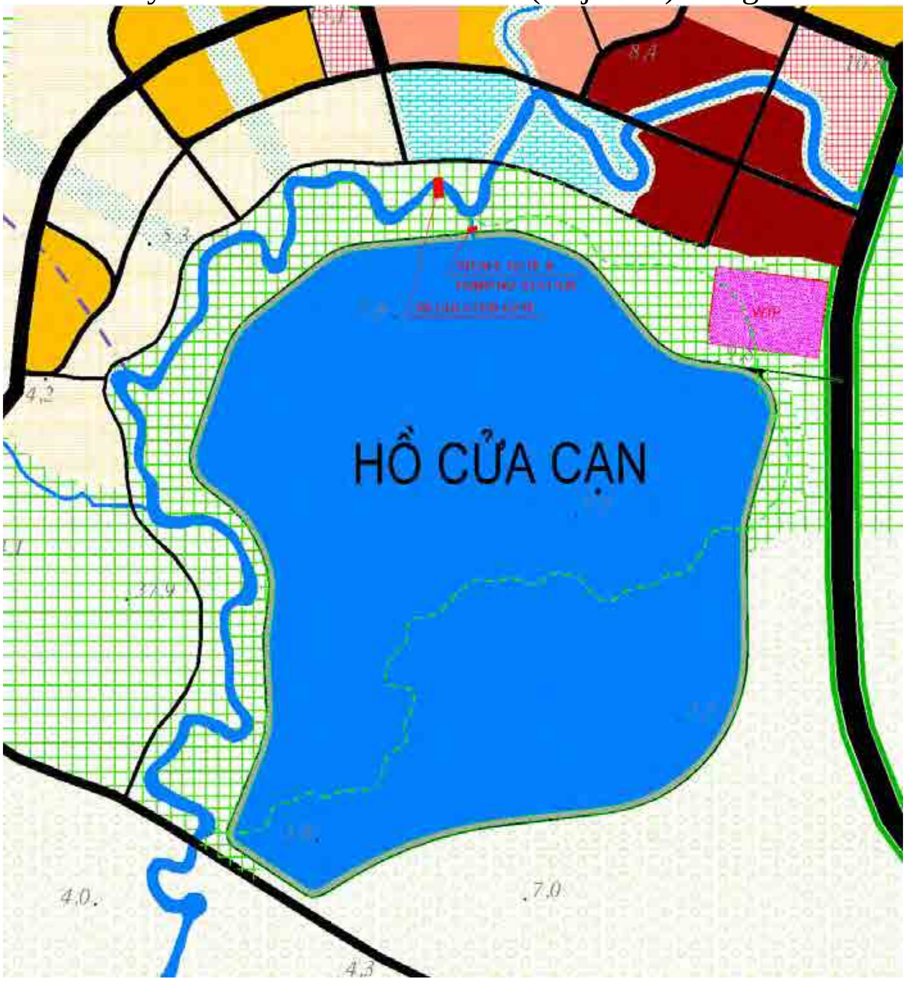
To overcome the above disadvantages, Cua Can reservoir is allocated towards a direction that it can keep the original status of Cua can river run-off; the reservoir will be built next to the river and the reservoir water is taken through the intake canal and pump. The reservoir is designed and constructed in two phases to meet the water demand.

In Phase 1: the reservoir is designed to retrieve a gravity water flow. To accomplish a gravity water flow, the water level in the reservoir should be lower than level of river bottom at the White Bridge of +4.18m. To facilitate, It is proposed to choose the normal water levels of the reservoir in phase 1 of +4.00 m.

Layout of Cua Can reservoirs (adjusted) in below.

Figure 3-2

Layout of Cua Can reservoirs ( adjusted) in figure




Adjust the Cua Can reservoir layout as above limit the disadvantages as in the Master Plan layout. However, some disadvantages of this layout are:

- While the water level in the reservoir is high, It requires to pump water into the reservoir resulting so large operating costs. This drawback can be partly overcome by building a large intake culvert to get water during heavy flood to reduce quantity of water haved to pumped or it can pump water directly from the river to supply for the water treatment plant, the rest lack of water will be taken from the reservoir.
- The reservoir area encroaches a part (about 60 hectares) of farmland in compared with the Master Plan. However, tree planting area has tantamountly increased in areas along Cua Can river.

From the above analysis, It is suggested to select options for allocating Cua Can reservoirs layout (adjustment) as in Figure 3 .2 as a basis for design calculations.

## 3.2 A Capcity of the Cua Can Reservoir

### *3.2.1 Capacity of the reservoir in the Mater Plan*

According to the Adjusted Master Plan to construct the Phu Quoc Island prepared by the South Urban- Rural Planning Sub-Institute in June, 2010, The capacity of Cua can reservoir is proposed as follows:

***The Cua Can Reservoirs: to build a Cua can reservoir with a capacity of W = 15 million m3 and the Water Supply Treatment Plant of Q = 50,000 m3/day. This is the biggest reservoir on the island, and that it is the main raw water supply source.***

### *3.2.2The Reservoir Function*

To supply water for the development of Phu Quoc island district, the functions of Cua reservoirs are determined as follows:

- Supply raw water for the WTP with a capacity of phase I of 21,000 $m^3$/and phase II of 52,500 $m^3$/day.
- Discharge water to maintain environmental flow and improve the basin environment in dry season.

## 3.3 Dertermining Volume of the Reservoir

### *3.3.1 Alternatives for the reservoir water level*

As analyzed above, in order not to cause flooding in Cua can Reservoir upstream area (of the National Forests), the water level in the Cua Can river must be lower than +4.18 m. So, It is able to get a gravity flow into the reservoir, the water level in the reservoir should be lower than an elevation of +4.18 m. Under these conditions, It is proposed to raise water level in the reservoir up to +4m and then a depth of water column in front of the intake culvert is around 2.5m which satify conditions for pumping water or gravity intaking water. Because the reservoir locates next the river and that the water getting to the reservoir by gravity flow through intake culvert or by pump can be completedly changed with no affects to the river water level and cannot cause the flooding in the reservoir upstream areas. For mentioned above reasons, the following alternatives on the reservoir water level could be considered:

***Alternative1:***

Phase 1: the reservoir water level in Phase 1 is kept in a level that it can obtain a gravity water flow. With the designing objective in phase I not to use the pump to get water into the Reservoir, the normal water levels in the reservoir of phase 1 must get lower than +4.18m. It is proposed to choose the reservoir's normal water level in phase 1 of +4.0m;

In phase 2: the reservoir's normal water level can be determined from the water balance; in this phase, the water demand increases and it needs to use pumps to get water into the reservoir when the water level in the reservoir is higher than +4. 0 m. In this alternative, operating costs will partially reduce because it does not require pumping water if the water level in the reservoir is lower than +4.00 m; However, It requires a huge cost for the soil excavation. Furthermore, if the elevation of reservoir bottom reaches +1.80 m (see water balance), all the reservoir bottom area will lie on a sandy soil layer having a strong water permeability to cause water loss and therefore, It needs to cover a seepage prevention clay layer with a thickness of about 1 m to the costs of the project construction.

***Alternative 2:*** the reservoir bottom is proposed to lay above the natural clay soil layer with a thickness of at least 1m; as presented in the strada log, in order to get a clay soil layer of 1m thickness, the reservoir bottom must be at elevation of +5.0m. if the reservoir bottom has an elvation of +5.0m, the depth of soil to be excavated will reduce 3.2m equivalent to about 6 millions m3 of filled soil in compared to the alternative 1 and it also do not require a clay soil layer of 1 m thickness to cover the reservoir bottom to prevent seepage. However this alternative has a demerit that water must be compltedly pumped to the reservoir.

***Alternative 3:*** To construct the reservoir by damming up the reservoir banks and no digging deep into the ground. It accordingly just takes the sufficient quantity of soil inside the reservoir bed to dam up the reservoir bank and does not dig deep into the reservoir ground any more. In this alternative, there are no unwanted mass of excavated soil and completedly solve the problem of finding out the area to dump the excess quantiy of excavated soil that is a thoroughly unsolved issue in alternative 1 & 2. However this alternative has demerits not only that water is completedly pumped to the reservoir but also the levels of water in the reservoir and dam crest are too high aboved the ouside natural ground elevation causing a loss of regional landscape. For this alternative, the reservoir bed can be flattened to facilitate the exploitation of the reservoir during the operation. It it proposed to bulldoze the reservoid bed to elevation of +7.0m. Quantiy of excavated soil to level the reservoir bed is estimated of arround 1 million m3.

Table 3.1 – Comparision on Construction cost of three different altetnatives

| **Alternative** | **Construction cost (dong)** | **Remark** |
|---|---|---|
| 1 | 592.864.686.000 | Largly impacted by a distance of soil transportation due to a huge quantitiy of waste soil. |
| 2 | 398.027.630.000 | Averagetedly influenced by the distance of waste land transportation due to a medium mass of waste soil. |
| 3 | 294.424.752.000 | Not affected by the distance of waste soil transportation as there is no ecxcess soil. |

***Analyses to select the alternativeof water level in the reservoir*** :

<u>*For Alternative 3:*</u> if simply comparing between the construction costs of the three diefferent alternatives, it is easily to realise that the alternative 3 is the most ecenomical and obtaines no risks on the transportation of waste soil to a junk yard in the case it cannot combine with another projects which need the excavated soil for their project ground filling at the same times. Considering on the aestheticworks aspect, It can see the dam crest elevation of the alternative 3 of +14.00 m (top of wall +15.20 m) is too much higher compared to the natural ground level of surrounding urban areas and does not create the beautifull environmental landscape for urban areas.

*<u>For Alternative 2:</u>* the execess excavated volume of this alternative of about 5.3 million m3 requires a cost of 35% higher the cost of alternative 1. However, this scenario has a dam crest elevation of+12.00 m (top of wall+13.2 m), 2m higher than urban area and higher the checked flood level of +11.14 m and sufficient free space to overcome flooding. Unwanted soil by digging reservoirs can be used to raise the ground of urban areas up to a level aboved the checked flood level and also create beautiful environmental scenery overlooking to the lake.

*<u>For Alternative 1:</u>* this alternative has a very expensive construction cost and further more, the seepage prevention for the reservoir bottom applied on a large area can lead to the risk of water loss and difficult to execute the work; For above reasons, the proposal should not be chosen

With above analysis, in this stage, It is proposed to choose the aternative 2 for calculations of other items'design. In a later stage, It needs to consider further aternatives on the reservoir bottom levels varying from +5.0m to +7.0m to find out the most reasonale aternative including a consideration of residual soil stored yard in the proposed alternatives.

### *3.3.2 The Performance curve of the reservoir bed*

The Performance Curve of the reservoir bed depends on water level in the reservoir and its digging depth. The reservoir Performance Curve is established by the process of calculating a balance of water. The result of the Reservoir performance curve is shown in the below charts.

Figure 3-3: the Performance Curve of the Reservoir Bed ( Alternative 1)

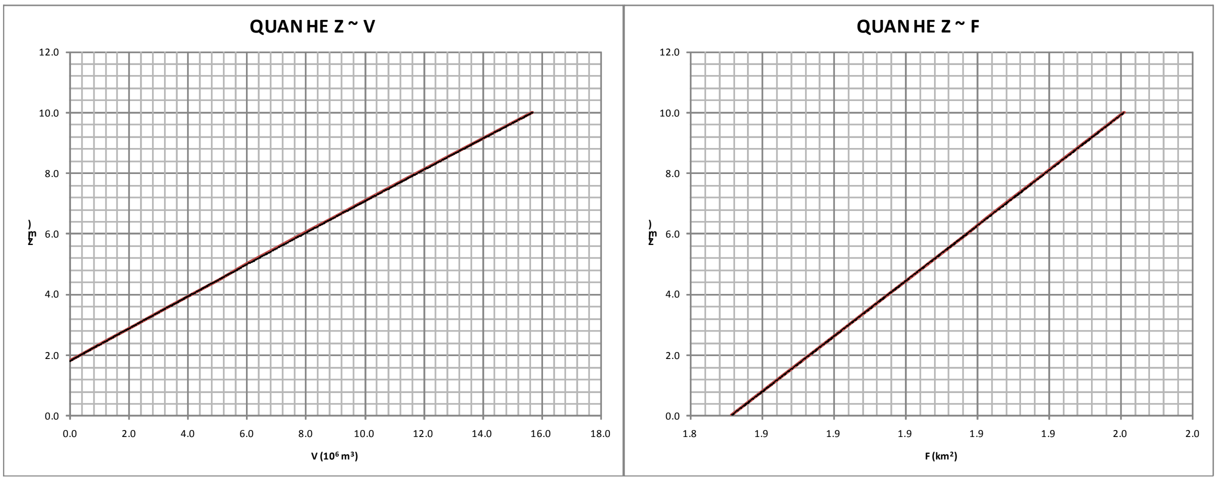



Figure 3-4: the Performance Curve of the Reservoir Bed ( Alternative 2)

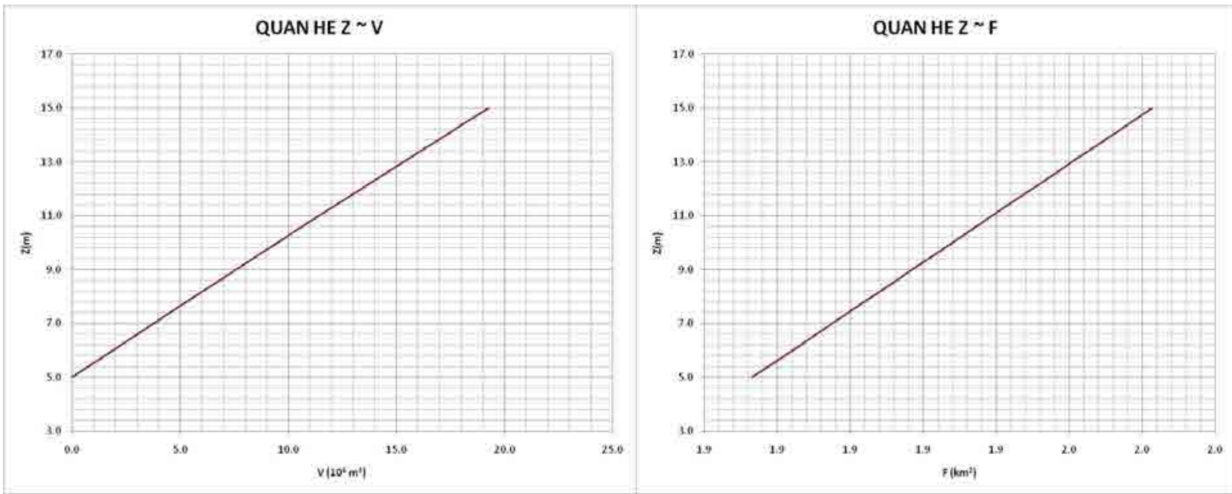
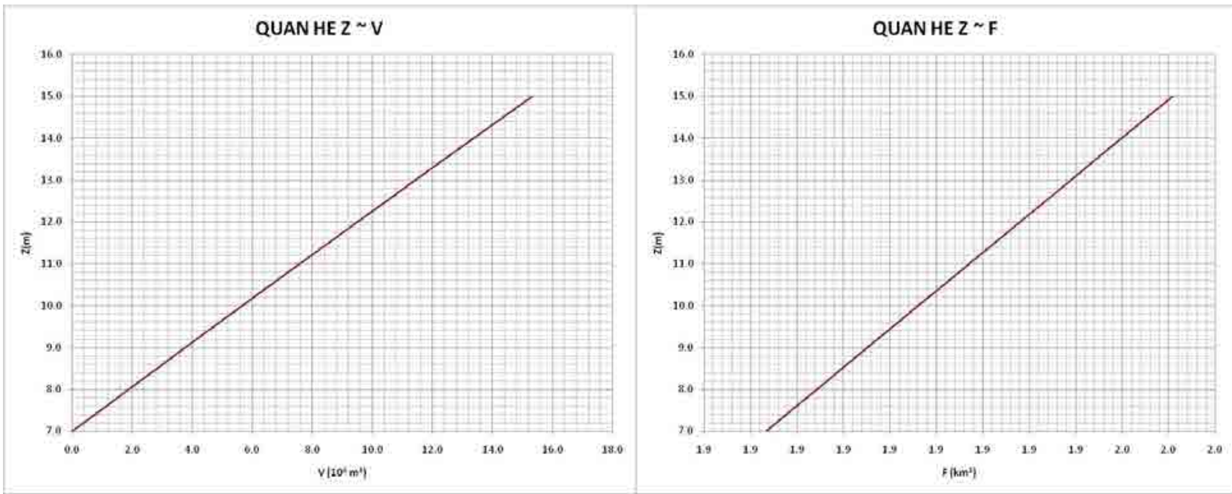



Figure 3-5: the Performance Curve of the Reservoir Bed ( Alternative 3)




### *3.3.3The Sediments and Dead Water Level of The Reservoir*

Table 3.1 – Sediment Volume

| No | Specifics | Unit | Phase 1 | Phase 2 |
|---|---|---|---|---|
| A | **Calculation of sediment volume** | | | |
| 1 | **The total amount of sediment to the reservoir every year (According to Meteo-hyrological reports)** | | | |
| | $V_{LL}$: Annual Amount of suspended solid. | m³/ year | 24.364 | 24.364 |
| | $V_d$: Annual Amount of bottom settled sediment. | m³/ year | 2.274 | 2.274 |
| | $V_{tm}$: Amount of vegetation | m³/ year | 266 | 266 |
| 2 | **Volume of mud and sand**<br>**$Vc = (V_{LL} + V_d*T) + 5*V_{tm}$**<br>$T_1$ = 10 years is the exploitation period of Phase 1<br>$T_2$ = 50 year is the life-span of the reservoir | million m³ | 0,268 | 1,33 |
| 3 | **Quantity of sediment – choose Vc(*)**<br>Vc = 50%*V'c | million m³ | **0,134** | **0,67** |
| **B** | **Dead water level of alternative 1** | | | |

| | | | | |
|---|---|---|---|---|
| | Sediment ( mud and sand) water level ($\nabla_{bc}$) | m | 1,87 | 2,16 |
| | Dead water level ($\nabla_c$) = $\nabla_{bc}$ + 0,5 | m | 2,40 | 2,70 |
| | Dead volume (Vc) | Million m³ | 1,12 | 1,69 |
| **C** | **Dead water level of alternative 2** | | | |
| | Sediment ( mud and sand) water level ($\nabla_{bc}$) | m | 5,07 | 5,36 |
| | Dead water level ($\nabla_c$) = $\nabla_{bc}$ + 0,5 | m | 5,60 | 5,90 |
| | Dead volume (Vc) | Million m³ | 1,13 | 1,69 |
| **D** | **Dead water level of alternative 3** | | | |
| | Sediment ( mud and sand) water level ($\nabla_{bc}$) | m | 7,07 | 7,36 |
| | Dead water level ($\nabla_c$) = $\nabla_{bc}$ + 0,5 | m | 7,60 | 7,90 |
| | Dead volume (Vc) | Million m³ | 1,13 | 1,69 |

*Note:*

- *Because a total water demand is about 36 million m3 equal to 1/2 volume of inflow water of 72 million m3 and considering on the safety factor, It should be suggested that the amount of natural sludge and sand settled down in the reservoir is equal to 50% of the total sediment quantity.*
- *To ensure the conditions for sufficient quatity of water to supply for Environmental maitenance flow and Water Treatment Plant It is suggested that dead water level in the reservor is higher the sediment water level of 0.5 m*
- *The dead water levels in the reservoir are rounded number.*

### *3.3.4 Water supply Demands*

*1. Domestic use and other economical branches water Demands.*

In phase 1: The Quantity of raw water of the WTP taken from the Cua Can reservoir is Q= 21,000m3/day.

In phase 2 : The capacity of WTP is 50,000m3/day and the water demand of WTP itself reaches 5% of the WTP capacity that the total quantity of raw water exploited from the Cua Can reservoir Q = 52,500$m^3$/day

*2. Environmental maintenance flow.*

According to some Water Reservoir Projects approved the Ministry of Agriculture and Rural Development (for example: the Water Reservoir Project in Mong Village, Nghe An province), a discharge volume to maintain the environmental flow should be at least equal to the exhautest flow in the exhautest months of application traffic frequency of P = 90%. Based on the hydrological calculations, in the March, the river flow is the smallest one; with a frequency P = 90%, River’s flow gets 0.237 m3/s. Thus, the minimum discharge volume to maintain the environmental flow is 0.237 m3/s

*3. Summarizing water demeands*

From the aboved calculations on water demand, Monthly water demands show in the following table.

Table 3.2–Water demand of phase I

| No | Month | days | Water Demand | | | | Total |
|---|---|---|---|---|---|---|---|
| | | | Domestic | | nvironmental maintenan flow | | |
| | | | Q (m³/day) | W ($10^6$m³) | Q (m³/day) | W ($10^6$m³) | W ($10^6$m³) |
| 1 | I | 31 | 21,000 | 0.651 | 0.237 | 0.635 | 1.286 |
| 2 | II | 28 | 21,000 | 0.588 | 0.237 | 0.573 | 1.161 |
| 3 | III | 31 | 21,000 | 0.651 | 0.237 | 0.635 | 1.286 |
| 4 | IV | 30 | 21,000 | 0.630 | 0.237 | 0.614 | 1.244 |
| 5 | V | 31 | 21,000 | 0.651 | 0.237 | 0.635 | 1.286 |
| 6 | VI | 30 | 21,000 | 0.630 | 0.237 | 0.614 | 1.244 |
| 7 | VII | 31 | 21,000 | 0.651 | 0.237 | 0.635 | 1.286 |
| 8 | VIII | 31 | 21,000 | 0.651 | 0.237 | 0.635 | 1.286 |
| 9 | IX | 30 | 21,000 | 0.630 | 0.237 | 0.614 | 1.244 |
| 10 | X | 31 | 21,000 | 0.651 | 0.237 | 0.635 | 1.286 |
| 11 | XI | 30 | 21,000 | 0.630 | 0.237 | 0.614 | 1.244 |
| 12 | XII | 31 | 21,000 | 0.651 | 0.237 | 0.635 | 1.286 |
| **Total** | | **365** | | **7.67** | | **7.47** | **15.14** |

Table 3.3 - Water demand of phase II

| No. | month | days | Water Demands | | | | Total |
|---|---|---|---|---|---|---|---|
| | | | Domestic | | vironmental maintenai flow | | |
| | | | Q (m³/day) | W ($10^6$m³) | Q (m³/day) | W ($10^6$m³) | W ($10^6$m³) |
| 1 | I | 31 | 52,500 | 1.628 | 0.237 | 0.635 | 2.262 |
| 2 | II | 28 | 52,500 | 1.470 | 0.237 | 0.573 | 2.043 |
| 3 | III | 31 | 52,500 | 1.628 | 0.237 | 0.635 | 2.262 |
| 4 | IV | 30 | 52,500 | 1.575 | 0.237 | 0.614 | 2.189 |
| 5 | V | 31 | 52,500 | 1.628 | 0.237 | 0.635 | 2.262 |
| 6 | VI | 30 | 52,500 | 1.575 | 0.237 | 0.614 | 2.189 |
| 7 | VII | 31 | 52,500 | 1.628 | 0.237 | 0.635 | 2.262 |
| 8 | VIII | 31 | 52,500 | 1.628 | 0.237 | 0.635 | 2.262 |
| 9 | IX | 30 | 52,500 | 1.575 | 0.237 | 0.614 | 2.189 |
| 10 | X | 31 | 52,500 | 1.628 | 0.237 | 0.635 | 2.262 |
| 11 | XI | 30 | 52,500 | 1.575 | 0.237 | 0.614 | 2.189 |
| 12 | XII | 31 | 52,500 | 1.628 | 0.237 | 0.635 | 2.262 |
| **Total** | | **365** | | **19.16** | | **7.47** | **26.64** |

### *3.3.5 Calculation of the Cua Can Reservoir Regulation*

Based on water demands and volume of inlet flow to ensure the frequency of P = 95%, calculating regulation of the reservoir by a typical method. The input data for water balance calculations include :

- The Reservoir evaporation obtained from hydrological data in theTable 2.22
- Inlet flow obtained from hydrological data in the Table 2.31

- Water demand shows in the Table 3.2& Bảng 3.3.Loss of a normal permeability of the Reservoir is from 2% to 5% of the reservoir capacity; However, due to characteristics of the reservoir built by damming up around the reservoir and geological conditions of the area having a bottom sand soil layer with high permeability, it is suggested that it should get a loss of the reservoi permeability of 10% of reservoir capacity.

Table 3.4 – Synthesizing calculation results of reservoir regulation

| No | Parameters | Unit | Alternative 1 | | Alternative 2 | | Alternative 3 | |
|---|---|---|---|---|---|---|---|---|
| | | | Phase 1 | Phase 2 | Phase 1 | Phase 2 | Phase 1 | Phase 2 |
| 1 | Reservoir bottom elevation | m | 1,80 | 1,80 | 5,0 | 5,00 | 7,00 | 7,00 |
| 2 | Dead water level | m | 2,40 | 2,70 | 5,60 | 5,90 | 7,60 | 7,90 |
| 3 | Normal ware level | m | 4,00 | 7,30 | 7,20 | 10,50 | 9,20 | 12,50 |
| 4 | Dead volume: $W_c$ | $10^6$ $m^3$ | 1,12 | 1,69 | 1,13 | 1,69 | 1,13 | 1,69 |
| 5 | Total capacity: $W_{tp}$ | $10^6$ $m^3$ | 4,14 | 10,46 | 4,15 | 10,47 | 4,15 | 10,47 |
| 6 | Effective capacity: $W_{hi}$ | $10^6$ $m^3$ | 3,02 | 8,77 | 3,02 | 8,77 | 3,02 | 8,77 |

Table 3.5 –the Reservoir regulation of phase 1 ( Alternative1)

| Month | $Q_{inflow}$ m³/s P95% | $W_{inflow}$ $10^6$m³ P95% | $W_{require}$ $10^6$ m³ | Excluding water loss | | | | | | Water loss | | | | | Including water loss | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Water balance | | V $10^6$ m³ | F km² | $V_{tb}$ $10^6$ m³ | $F_{tb}$ km² | Evaporation | | Seepage | | Total $10^6$ m³ | Water balance | | V $10^6$ m³ | $W_{release}$ $10^6$m³ | Z (m) |
| | | | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | | | | | ΔZ mm | Wz $10^6$ m³ | Criteria | $W_{th}$ $10^6$ m³ | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) |
| 5 | | 1.92 | | | | | | | | | | | | | | | 1.55 | | 2.63 |
| 6 | 1.628 | 4.22 | 1.24 | 2.98 | | 2.73 | 1.89 | 1.93 | 1.88 | 45.3 | 0.085 | | 0.19 | 0.28 | 2.70 | | 3.82 | 0 | 3.83 |
| 7 | 3.406 | 9.12 | 1.29 | 7.84 | | 2.73 | 1.89 | 2.73 | 1.89 | 42.6 | 0.080 | | 0.27 | 0.35 | 7.48 | | 3.97 | 7.33 | 3.91 |
| 8 | 4.345 | 11.64 | 1.29 | 10.35 | | 2.73 | 1.89 | 2.73 | 1.89 | 43.4 | 0.082 | | 0.27 | 0.35 | 10.00 | | 3.97 | 10.00 | 3.91 |
| 9 | 5.575 | 14.45 | 1.24 | 13.21 | | 2.73 | 1.89 | 2.73 | 1.89 | 38.8 | 0.073 | | 0.27 | 0.35 | 12.86 | | 3.97 | 12.86 | 3.91 |
| 10 | 6.866 | 18.39 | 1.29 | 17.10 | | 2.73 | 1.89 | 2.73 | 1.89 | 38.1 | 0.072 | Take 10% of Average Volume | 0.27 | 0.34 | 16.76 | | 3.97 | 16.76 | 3.91 |
| 11 | 2.723 | 7.06 | 1.24 | 5.81 | | 2.73 | 1.89 | 2.73 | 1.89 | 59.7 | 0.113 | | 0.27 | 0.39 | 5.43 | | 3.97 | 5.43 | 3.91 |
| 12 | 1.042 | 2.79 | 1.29 | 1.51 | | 2.73 | 1.89 | 2.73 | 1.89 | 77.3 | 0.146 | | 0.27 | 0.42 | 1.09 | | 3.97 | 1.09 | 3.91 |
| 1 | 0.478 | 1.28 | 1.29 | | 0.01 | 2.72 | 1.89 | 2.73 | 1.89 | 62.8 | 0.119 | | 0.27 | 0.39 | | 0.40 | 3.58 | 0 | 3.70 |
| 2 | 0.255 | 0.62 | 1.16 | | 0.54 | 2.18 | 1.88 | 2.45 | 1.89 | 52.2 | 0.098 | | 0.25 | 0.34 | | 0.89 | 2.69 | 0 | 3.23 |
| 3 | 0.202 | 0.54 | 1.29 | | 0.74 | 1.43 | 1.88 | 1.81 | 1.88 | 56.6 | 0.106 | | 0.18 | 0.29 | | 1.03 | 1.66 | 0 | 2.68 |
| 4 | 0.361 | 0.94 | 1.24 | | 0.31 | 1.12 | 1.88 | 1.28 | 1.88 | 50.6 | 0.095 | | 0.13 | 0.22 | | 0.53 | 1.12 | 0 | 2.40 |
| 5 | 0.715 | 1.92 | 1.29 | 0.63 | | 1.12 | 1.88 | 1.12 | 1.88 | 48.7 | 0.091 | | 0.11 | 0.20 | 0.43 | | 1.55 | 0 | 2.63 |
| Total | | 72.96 | **15.14** | **59.42** | **1.60** | | | | | **616.1** | **1.161** | | **2.77** | **3.93** | **56.74** | **2.85** | | **53.46** | **3.91** |

Normal water level selected for phase 1: **4.00m**

Total reservoir volume for phase 1: **4.14x$10^6$m³**

Table 3.1 – the Reservoir regulation of phase 2 ( Alternative1)

| Month | $Q_{inflow}$ m³/s P95% | $W_{inflow}$ $10^6$m³ P95% | $W_{require}$ $10^6$ m³ | Excluding water loss | | | | | | Water loss | | | | | Including water loss | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Water balance | | V $10^6$ m³ | F km² | $V_{tb}$ $10^6$ m³ | $F_{tb}$ km² | Evaporation | | Seepage | | Total $10^6$ m³ | Water balance | | V $10^6$ m³ | $W_{release}$ $10^6$m³ | Z (m) |
| | | | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | | | | | DZ mm | Wz $10^6$ m³ | Criteria | $W_{th}$ $10^6$ m³ | | DW+ $10^6$ m³ | DW- $10^6$ m³ | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) |
| 5 | | 1.92 | | | | | | | | | | | | | | | 1.69 | | 2.70 |
| 6 | 1.628 | 4.22 | 2.19 | 2.03 | | 3.72 | 1.89 | 2.70 | 1.89 | 45.3 | 0.085 | Getting 10% of average Dry Volume | 0.27 | 0.36 | 1.67 | | 3.36 | 0 | 3.59 |
| 7 | 3.406 | 9.12 | 2.26 | 6.86 | | 6.89 | 1.91 | 5.30 | 1.90 | 42.6 | 0.081 | | 0.53 | 0.61 | 6.25 | | 9.61 | 0 | 6.86 |
| 8 | 4.345 | 11.64 | 2.26 | 9.38 | | 6.89 | 1.91 | 6.89 | 1.91 | 43.4 | 0.083 | | 0.69 | 0.77 | 8.60 | | 10.40 | 7.82 | 7.27 |
| 9 | 5.575 | 14.45 | 2.19 | 12.26 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.8 | 0.074 | | 0.69 | 0.76 | 11.50 | | 10.40 | 11.50 | 7.27 |
| 10 | 6.866 | 18.39 | 2.26 | 16.13 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.1 | 0.073 | | 0.69 | 0.76 | 15.37 | | 10.40 | 15.37 | 7.27 |
| 11 | 2.723 | 7.06 | 2.19 | 4.87 | | 6.89 | 1.91 | 6.89 | 1.91 | 59.7 | 0.114 | | 0.69 | 0.80 | 4.07 | | 10.40 | 4.07 | 7.27 |
| 12 | 1.042 | 2.79 | 2.26 | | -0.53 | 7.42 | 1.91 | 7.15 | 1.91 | 77.3 | 0.148 | | 0.72 | 0.86 | | 0.33 | 10.07 | 0 | 7.10 |
| 1 | 0.478 | 1.28 | 2.26 | | 0.98 | 6.44 | 1.91 | 6.93 | 1.91 | 62.8 | 0.120 | | 0.69 | 0.81 | | 1.79 | 8.27 | 0 | 6.16 |
| 2 | 0.255 | 0.62 | 2.04 | | 1.43 | 5.01 | 1.90 | 5.72 | 1.90 | 52.2 | 0.099 | | 0.57 | 0.67 | | 2.10 | 6.17 | 0 | 5.07 |
| 3 | 0.202 | 0.54 | 2.26 | | 1.72 | 3.29 | 1.89 | 4.15 | 1.90 | 56.6 | 0.107 | | 0.42 | 0.52 | | 2.24 | 3.93 | 0 | 3.89 |
| 4 | 0.361 | 0.94 | 2.19 | | 1.25 | 2.04 | 1.88 | 2.66 | 1.89 | 50.6 | 0.095 | | 0.27 | 0.36 | | 1.62 | 2.31 | 0 | 3.03 |
| 5 | 0.715 | 1.92 | 2.26 | | 0.35 | 1.69 | 1.88 | 1.86 | 1.88 | 48.7 | 0.092 | | 0.19 | 0.28 | | 0.63 | 1.69 | 0 | 2.70 |
| Total | | 72.96 | **26.64** | **51.52** | **5.20** | | | | | **616.1** | **1.172** | | **6.41** | **7.58** | **47.46** | **8.71** | | **38.74** | **7.27** |

Normal water level selected for phase 2 **:7.30m**

Total reservoir volume for phase 2: **10.46x$10^6$m³**

Table 3.2 – The reservoir Regulation in phase 1 (Alternative 2)

| Month | $Q_{inflow}$ m³/s P95% | $W_{inflow}$ $10^6$m³ P95% | $W_{require}$ $10^6$ m³ | Excluding water loss | | | | | | Water loss | | | | | Including water loss | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Water balance | | | | | | Evaporation | | Seepage | | | Water balance | | | | |
| | | | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | V $10^6$ m³ | F km² | $V_{tb}$ $10^6$ m³ | $F_{tb}$ km² | ΔZ mm | Wz $10^6$ m³ | Criteria | $W_{th}$ $10^6$ m³ | Total $10^6$ m³ | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | V $10^6$ m³ | $W_{release}$ $10^6$m³ | Z (m) |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) |
| 5 | | 1.92 | | | | | | | | | | | | | | | 1.69 | | 2.70 |
| 6 | 1.628 | 4.22 | 2.19 | 2.03 | | 3.72 | 1.89 | 2.70 | 1.89 | 45.3 | 0.085 | | 0.27 | 0.36 | 1.67 | | 3.36 | 0 | 3.59 |
| 7 | 3.406 | 9.12 | 2.26 | 6.86 | | 6.89 | 1.91 | 5.30 | 1.90 | 42.6 | 0.081 | | 0.53 | 0.61 | 6.25 | | 9.61 | 0 | 6.86 |
| 8 | 4.345 | 11.64 | 2.26 | 9.38 | | 6.89 | 1.91 | 6.89 | 1.91 | 43.4 | 0.083 | | 0.69 | 0.77 | 8.60 | | 10.40 | 7.82 | 7.27 |
| 9 | 5.575 | 14.45 | 2.19 | 12.26 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.8 | 0.074 | | 0.69 | 0.76 | 11.50 | | 10.40 | 11.50 | 7.27 |
| 10 | 6.866 | 18.39 | 2.26 | 16.13 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.1 | 0.073 | take 10% of average Volume | 0.69 | 0.76 | 15.37 | | 10.40 | 15.37 | 7.27 |
| 11 | 2.723 | 7.06 | 2.19 | 4.87 | | 6.89 | 1.91 | 6.89 | 1.91 | 59.7 | 0.114 | | 0.69 | 0.80 | 4.07 | | 10.40 | 4.07 | 7.27 |
| 12 | 1.042 | 2.79 | 2.26 | | -0.53 | 7.42 | 1.91 | 7.15 | 1.91 | 77.3 | 0.148 | | 0.72 | 0.86 | | 0.33 | 10.07 | 0 | 7.10 |
| 1 | 0.478 | 1.28 | 2.26 | | 0.98 | 6.44 | 1.91 | 6.93 | 1.91 | 62.8 | 0.120 | | 0.69 | 0.81 | | 1.79 | 8.27 | 0 | 6.16 |
| 2 | 0.255 | 0.62 | 2.04 | | 1.43 | 5.01 | 1.90 | 5.72 | 1.90 | 52.2 | 0.099 | | 0.57 | 0.67 | | 2.10 | 6.17 | 0 | 5.07 |
| 3 | 0.202 | 0.54 | 2.26 | | 1.72 | 3.29 | 1.89 | 4.15 | 1.90 | 56.6 | 0.107 | | 0.42 | 0.52 | | 2.24 | 3.93 | 0 | 3.89 |
| 4 | 0.361 | 0.94 | 2.19 | | 1.25 | 2.04 | 1.88 | 2.66 | 1.89 | 50.6 | 0.095 | | 0.27 | 0.36 | | 1.62 | 2.31 | 0 | 3.03 |
| 5 | 0.715 | 1.92 | 2.26 | | 0.35 | 1.69 | 1.88 | 1.86 | 1.88 | 48.7 | 0.092 | | 0.19 | 0.28 | | 0.63 | 1.69 | 0 | 2.70 |
| Total | | 72.96 | **26.64** | **51.52** | **5.20** | | | | | **616.1** | **1.172** | | **6.41** | **7.58** | **47.46** | **8.71** | | **38.74** | **7.27** |

Normal water level selected for phase 2 **7.30m**

Total reservoir volume for phase 2: **10.46x10$^6$m³**

## Table 3.8 – The reservoir Regulation in phase 2 (Alternative 2)

| Month | $Q_{inflow}$ m³/s P95% | $W_{inflow}$ $10^6$m³ P95% | $W_{require}$ $10^6$ m³ | Excluding water loss | | | | | | Water loss | | | | | Including water loss | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Water balance | | V $10^6$ m³ | F km² | $V_{tb}$ $10^6$ m³ | $F_{tb}$ km² | Evaporation | | Seepage | | $10^6$ m³ | Water balance | | V $10^6$ m³ | $W_{release}$ $10^6$m³ | Z (m) |
| | | | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | | | | | ΔZ mm | Wz $10^6$ m³ | Criteria | $W_{th}$ $10^6$ m³ | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) |
| 5 | | 1.92 | | | | | | | | | | | | | | | 1.69 | | 5.90 |
| 6 | 1.628 | 4.22 | 2.19 | 2.03 | | 3.72 | 1.89 | 2.71 | 1.89 | 45.3 | 0.086 | | 0.27 | 0.36 | 1.67 | | 3.36 | 0 | 6.79 |
| 7 | 3.406 | 9.12 | 2.26 | 6.86 | | 6.89 | 1.91 | 5.31 | 1.90 | 42.6 | 0.081 | | 0.53 | 0.61 | 6.25 | | 9.61 | 0 | 10.06 |
| 8 | 4.345 | 11.64 | 2.26 | 9.38 | | 6.89 | 1.91 | 6.89 | 1.91 | 43.4 | 0.083 | | 0.69 | 0.77 | 8.60 | | 10.40 | 7.81 | 10.47 |
| 9 | 5.575 | 14.45 | 2.19 | 12.26 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.8 | 0.074 | | 0.69 | 0.76 | 11.50 | | 10.40 | 11.50 | 10.47 |
| 10 | 6.866 | 18.39 | 2.26 | 16.13 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.1 | 0.073 | Getting 10% of Average Volume | 0.69 | 0.76 | 15.37 | | 10.40 | 15.37 | 10.47 |
| 11 | 2.723 | 7.06 | 2.19 | 4.87 | | 6.89 | 1.91 | 6.89 | 1.91 | 59.7 | 0.114 | | 0.69 | 0.80 | 4.07 | | 10.40 | 4.07 | 10.47 |
| 12 | 1.042 | 2.79 | 2.26 | | -0.53 | 7.42 | 1.92 | 7.16 | 1.91 | 77.3 | 0.148 | | 0.72 | 0.86 | | 0.34 | 10.07 | 0 | 10.29 |
| 1 | 0.478 | 1.28 | 2.26 | | 0.98 | 6.44 | 1.91 | 6.93 | 1.91 | 62.8 | 0.120 | | 0.69 | 0.81 | | 1.80 | 8.27 | 0 | 9.36 |
| 2 | 0.255 | 0.62 | 2.04 | | 1.43 | 5.01 | 1.90 | 5.73 | 1.91 | 52.2 | 0.100 | | 0.57 | 0.67 | | 2.10 | 6.18 | 0 | 8.27 |
| 3 | 0.202 | 0.54 | 2.26 | | 1.72 | 3.29 | 1.89 | 4.15 | 1.90 | 56.6 | 0.107 | | 0.42 | 0.52 | | 2.24 | 3.93 | 0 | 7.09 |
| 4 | 0.361 | 0.94 | 2.19 | | 1.25 | 2.04 | 1.89 | 2.66 | 1.89 | 50.6 | 0.096 | | 0.27 | 0.36 | | 1.62 | 2.32 | 0 | 6.23 |
| 5 | 0.715 | 1.92 | 2.26 | | 0.35 | 1.69 | 1.88 | 1.86 | 1.88 | 48.7 | 0.092 | | 0.19 | 0.28 | | 0.63 | 1.69 | 0 | 5.90 |
| Total | | 72.96 | **26.64** | **51.52** | **5.20** | | | | | **616.1** | **1.173** | | **6.41** | **7.58** | **47.45** | **8.71** | | **38.74** | **10.47** |

Normal water level selected for Phase 2 **10.50m**

Total reservoir volume for Phase 2: **10.47x$10^6$m³**

## Table 3.9 – The reservoir Regulation in phase 1 (Alternative 3)

| Month | $Q_{inflow}$ m³/s P95% | $W_{inflow}$ $10^6$m³ P95% | $W_{require}$ $10^6$ m³ | Excluding water loss | | | | | | Water loss | | | | | Including water loss | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Water balance | | V $10^6$ m³ | F km² | $V_{tb}$ $10^6$ m³ | $F_{tb}$ km² | Evaporation | | Seepage | | Total $10^6$ m³ | Water balance | | V $10^6$ m³ | $W_{release}$ $10^6$m³ | Z (m) |
| | | | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | | | | | ΔZ mm | Wz $10^6$ m³ | Criteria | $W_{th}$ $10^6$ m³ | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) |
| 5 | | 1.92 | | | | | | | | | | | | | | | 1.55 | | 7.83 |
| 6 | 1.628 | 4.22 | 1.24 | 2.98 | | 2.73 | 1.89 | 1.93 | 1.88 | 45.3 | 0.085 | take 10 percent-age of average volume | 0.19 | 0.28 | 2.70 | | 3.82 | 0.43 | 9.03 |
| 7 | 3.406 | 9.12 | 1.29 | 7.84 | | 2.73 | 1.89 | 2.73 | 1.89 | 42.6 | 0.080 | | 0.27 | 0.35 | 7.48 | | 3.97 | 7.33 | 9.11 |
| 8 | 4.345 | 11.64 | 1.29 | 10.35 | | 2.73 | 1.89 | 2.73 | 1.89 | 43.4 | 0.082 | | 0.27 | 0.35 | 10.00 | | 3.97 | 10.00 | 9.11 |
| 9 | 5.575 | 14.45 | 1.24 | 13.21 | | 2.73 | 1.89 | 2.73 | 1.89 | 38.8 | 0.073 | | 0.27 | 0.35 | 12.86 | | 3.97 | 12.86 | 9.11 |
| 10 | 6.866 | 18.39 | 1.29 | 17.10 | | 2.73 | 1.89 | 2.73 | 1.89 | 38.1 | 0.072 | | 0.27 | 0.34 | 16.76 | | 3.97 | 16.76 | 9.11 |
| 11 | 2.723 | 7.06 | 1.24 | 5.81 | | 2.73 | 1.89 | 2.73 | 1.89 | 59.7 | 0.113 | | 0.27 | 0.39 | 5.43 | | 3.97 | 5.43 | 9.11 |
| 12 | 1.042 | 2.79 | 1.29 | 1.51 | | 2.73 | 1.89 | 2.73 | 1.89 | 77.3 | 0.146 | | 0.27 | 0.42 | 1.09 | | 3.97 | 1.09 | 9.11 |
| 1 | 0.478 | 1.28 | 1.29 | | 0.01 | 2.72 | 1.89 | 2.73 | 1.89 | 62.8 | 0.119 | | 0.27 | 0.39 | | 0.40 | 3.58 | 0 | 8.90 |
| 2 | 0.255 | 0.62 | 1.16 | | 0.54 | 2.18 | 1.89 | 2.45 | 1.89 | 52.2 | 0.099 | | 0.25 | 0.34 | | 0.89 | 2.69 | 0 | 8.43 |
| 3 | 0.202 | 0.54 | 1.29 | | 0.74 | 1.43 | 1.88 | 1.81 | 1.88 | 56.6 | 0.107 | | 0.18 | 0.29 | | 1.03 | 1.66 | 0 | 7.88 |
| 4 | 0.361 | 0.94 | 1.24 | | 0.31 | 1.13 | 1.88 | 1.28 | 1.88 | 50.6 | 0.095 | | 0.13 | 0.22 | | 0.53 | 1.13 | 0 | 7.60 |
| 5 | 0.715 | 1.92 | 1.29 | 0.63 | | 1.13 | 1.88 | 1.13 | 1.88 | 48.7 | 0.092 | | 0.11 | 0.20 | 0.43 | | 1.55 | 0 | 7.83 |
| Total | | 72.96 | **15.14** | **59.42** | **1.60** | | | | | **616.1** | **1.162** | | **2.77** | **3.93** | **56.74** | **2.85** | | **53.46** | **9.11** |

Normal water level selected for Phase 1 **9.20m**

Total reservoir volume for Phase 1: **4.15x$10^6$m³**

Table 3.10– The reservoir Regulation in phase 2 (Alternative 3)

| Month | | | | Excluding water loss | Water loss | Including water loss |
|---|---|---|---|---|---|---|

| | $Q_{inflow}$ m³/s P95% | | | Water balance | | | | | | Evaporation | | Seepage | | | Water balance | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | V $10^6$ m³ | F km² | $V_{tb}$ $10^6$ m³ | $F_{tb}$ km² | ΔZ mm | Wz $10^6$ m³ | Criteria | $W_{th}$ $10^6$ m³ | Total $10^6$ m³ | ΔW+ $10^6$ m³ | ΔW- $10^6$ m³ | V $10^6$ m³ | $W_{release}$ $10^6$m³ | Z (m) |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) |
| 5 | | 1.92 | | | | | | | | | | | | | | | 1.69 | | 7.90 |
| 6 | 1.628 | 4.22 | 2.19 | 2.03 | | 3.72 | 1.89 | 2.71 | 1.89 | 45.3 | 0.086 | take 10 percent-age of average volume | 0.27 | 0.36 | 1.67 | | 3.36 | 0 | 8.79 |
| 7 | 3.406 | 9.12 | 2.26 | 6.86 | | 6.89 | 1.91 | 5.31 | 1.90 | 42.6 | 0.081 | | 0.53 | 0.61 | 6.25 | | 9.61 | 0 | 12.06 |
| 8 | 4.345 | 11.64 | 2.26 | 9.38 | | 6.89 | 1.91 | 6.89 | 1.91 | 43.4 | 0.083 | | 0.69 | 0.77 | 8.60 | | 10.40 | 7.81 | 12.47 |
| 9 | 5.575 | 14.45 | 2.19 | 12.26 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.8 | 0.074 | | 0.69 | 0.76 | 11.50 | | 10.40 | 11.50 | 12.47 |
| 10 | 6.866 | 18.39 | 2.26 | 16.13 | | 6.89 | 1.91 | 6.89 | 1.91 | 38.1 | 0.073 | | 0.69 | 0.76 | 15.37 | | 10.40 | 15.37 | 12.47 |
| 11 | 2.723 | 7.06 | 2.19 | 4.87 | | 6.89 | 1.91 | 6.89 | 1.91 | 59.7 | 0.114 | | 0.69 | 0.80 | 4.07 | | 10.40 | 4.07 | 12.47 |
| 12 | 1.042 | 2.79 | 2.26 | | -0.53 | 7.42 | 1.92 | 7.16 | 1.91 | 77.3 | 0.148 | | 0.72 | 0.86 | | 0.34 | 10.07 | 0 | 12.29 |
| 1 | 0.478 | 1.28 | 2.26 | | 0.98 | 6.44 | 1.91 | 6.93 | 1.91 | 62.8 | 0.120 | | 0.69 | 0.81 | | 1.80 | 8.27 | 0 | 11.36 |
| 2 | 0.255 | 0.62 | 2.04 | | 1.43 | 5.01 | 1.90 | 5.73 | 1.91 | 52.2 | 0.100 | | 0.57 | 0.67 | | 2.10 | 6.18 | 0 | 10.27 |
| 3 | 0.202 | 0.54 | 2.26 | | 1.72 | 3.29 | 1.89 | 4.15 | 1.90 | 56.6 | 0.107 | | 0.42 | 0.52 | | 2.24 | 3.93 | 0 | 9.09 |
| 4 | 0.361 | 0.94 | 2.19 | | 1.25 | 2.04 | 1.89 | 2.66 | 1.89 | 50.6 | 0.096 | | 0.27 | 0.36 | | 1.62 | 2.32 | 0 | 8.23 |
| 5 | 0.715 | 1.92 | 2.26 | | 0.35 | 1.69 | 1.88 | 1.86 | 1.88 | 48.7 | 0.092 | | 0.19 | 0.28 | | 0.63 | 1.69 | 0 | 7.90 |
| Total | | 72.96 | **26.64** | **51.52** | **5.20** | | | | | **616.1** | **1.173** | | **6.41** | **7.58** | **47.45** | **8.71** | | **38.74** | **12.47** |

Normal water level selected for phase 2 **12.50m**

Total reservoir volume for phase 2: **10.47x$10^6$m³**

## 3.4 Preliminary design of main works

### *3.4.1 Seepage prevention for The Water Reservoir*

According to results of the geological surveys, the clay (CL, CL1) soil layers are capable of waterproofing with permeable coefficient of K = 8.95x10-6 cm/s only appear on the ground surface, the bottom elevation of layer varies from +6.00 m to +1.00 m with the thickness varies from 0m to 6m. Below ground waterproofing soil layer is sandy clay layers of SC, SC1 or SC2 with strong permeable coefficient in the range from K = 3.44x10-4 cm / s to K = 1.05x10-3 cm / s. With a designed elevation of the reservoir bottom at +5.0 m, most area of the reservoir bottom will lie on clay layers of CL,CL1, SC1 or SC2 which have relatively strong seepage prevention capabilities and therefore, It does not require applying the construction measures to prevent seepage for the reservoir bottom. To prevent seepage for the bottom area of the lake (of about 40 ha area), technical solutions can be considered are:

- The bottom area will be lined with HDPE clothe or a thin clay soil layer below another backfill soil layer against floading. If it preliminarily selects the soil backfill layer of 0.5 m thick that the unit price for a square meter anti-seepage is 120,000 dong and the total estimate cost for the entire lake bottom area waterproofing is 48 billions of VND. These layers have prevented seepage very well, but requiring too expensive cost. Moreover, 80% of the reservoir bottom is covered with the natural anti-seepage clay soil layer, so this altinative is not suitable as it does not create a homogeneous material to prevent seepage for the reservoir bottom and the joints between the man-made layers with the natural clay soil layers are too difficult to ensure the quality of seepage prevention.
- Applying soil filled layer to prevent seepage. The soil layer thickness is determined under formula : $\delta \geq Z/[J_{cp}]$. With the difference of the water column in and out of the lake of Z = 5.5m (assuming that the underground water level is equal a level of the the lake's bottom) and allowable seepage gradient $[J_{cp}]$= 5-10, the thickness of anti-seepage clay soil layer will be from 1.1 m to 0.55m. For natural safety, it is proposed to choose a clay soil layer thickness of 1.0 m. At these conditions, a unit price for the anti-seepage is 30,000 dong/$m^2$ and the total estimated cost for the entire reservoir is 12 billions. This technical option is not able to absolutely prevent seepage flow; However It can limit permeability of the reservoir and the cost is cheaper as it can utilize clay soil source available in the reservoir bed and can overcome the demerit of linning the reservoir bottom with HDPE clothe.

Because water source is relatively abundant for getting water into the lake including amount of water seepage loss; It is suggested to select the anti- seepage option of constructing a layer of a clay soil up to 1.0m thick. The total filled soil mass covering about 360,000 m3.

### *3.4.2 Earth dam*

#### *a) Earth Dam Structure*

It needs to select structure of the earth dam as the following reasons:

- Utilising the soil excavated from the reservoir bed to dam up.

- It is sufficient volume of clay soil for dam up and this amount of clay can be taken right in the reservoir bed.
- Strata log of the soil area is suitable for earth dams as it has a small foundation stress, a capability to be under the deformation higher that other types of dam.

***b) The Dam Crest Level***

Because it is the earth dam, the mandatory condition is that the water must be controlled not to overflow the dam crest. Due to water only intaken to the reservoir that it calculates the dam crest level above the reservoir normal water level in phase1 & 2. The height (d) from the static water level to the dam top is calculated by the following formula:

$d = \Delta h + h_{sl} + a$

where :

$\Delta h$ – wave surchage

$h_{sl}$ – run-up

a – free board = 0,5m

***A height of Wave surcharge of Δh under QPTL C-1-78.***

$$\Delta h = 2x10^{-6}\frac{v^2D}{g.H}\cos\alpha$$

v: Wind velocity (m/s); $v_{4\%}$ = 41,7 m/s

D: Length of wind direction (m); D=1686 m

$\alpha$: Angle between wind direction and the reservoir center line; $\alpha = 0$

H: height of water colume in upstream side of dam to thedam bottom (m);

- Phase 1: H = 2.2m; $\Delta h_1$ = 0.271 m
- Phase 2: H = 5.5m; $\Delta h_2$ = 0.107 m

***Calculating a height of the run-up following the QPTL C-1-78 (frequency of 1%); $h_{sl1\%}$.***

$$h_{sl1\%} = k_1.k_2.k_3k_4.h_{1\%}$$

$$h_{1\%} = k_{1\%}.h_s$$

$k_1$, $k_2$ depends on material reinforecing dam slopes. Dam solpes reinforced by concrete slabs $k_1$=1.0; $k_2$ = 0.9.

$k_3$ looking up in the table No.7 (*QPTL C-1-78)*

$k_4$ finding in chart of figure No. 10 (*QPTL C-1-78)*

$k_{1\%}$ reffered from chart of figure No. 36 (*QPTL C-1-78)*

Table 3.11 – Calculation a height of wave surcharge

| Phase | $gD/v^2$ | $g\tau/v$ | $g.H/v^2$ | $g.\tau/v$ | $\tau$ | $g.h_s/v^2$ | $h_s$ | $\lambda$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 9.51 | 5081 | 0.012 | 0.45 | 1.91 | 0.0024 | 0.43 | 5.71 |
| 2 | 9.51 | 5081 | 0.032 | 0.65 | 2.76 | 0.0045 | 0.80 | 11.92 |

$g.\tau/v$; $g.h_s/v^2$ are values reffered from charts No.35 (*QPTL C-1-78)*

Table 3.12 – Calculation a height of run-up wave

| Phase | $k_{1\%}$ | $h_{1\%}$ | $\lambda/h_{1\%}$ | $k_1$ | $k_2$ | $k_3$ | $k_4$ | $h_{sl1\%}$ (m) |
|---|---|---|---|---|---|---|---|---|
| 1 | 2.07 | 0.88 | 6.49 | 1 | 0.9 | 1.4 | 1.80 | 2.00 |
| 2 | 2.07 | 1.65 | 7.22 | 1 | 0.9 | 1.4 | 1.85 | 3.85 |

***The Dam Crest Level in The Case of Without A Breakwater Wall***

Table 3.13 – a height of dam crest (exclusive Breakewater wall)

| Phase | Water level (m) | $\Delta h$ (m) | $h_{sl}$ (m) | a (m) | Dam crest (m) |
|---|---|---|---|---|---|
| 1 | 4.0 | 0.27 | 2.00 | 0.50 | 6.80 |
| 2 | 7.3 | 0.11 | 3.85 | 0.50 | 11.80 |

***The Dam Crest level in The Case of constructing A Breakwater Wall on Dam crest surface***

$d = \Delta h + \eta_c + h_1 + a$ ; $\eta_c = p_i/\xi.g$; $p_i = 0.7(1-a_1/a_r)p_u$;

$p_u = \xi.g.h_s(0.033\lambda/H+0.75)$; $\xi = 1/(m.(h_s/\lambda)^{0.5})$

$\eta_c = 0.7h_{s1\%}(0.033\lambda/H+0.75)(1-a_1/a_r)$

$\eta_c$ – a height of wave.

$h_1$ – a height from water level to foot of the breakwater wall

Table 3.14 –a height of wave

| Phase | $\xi$ | Pu | $a_1$ | $a_r$ | $P_i$ | $\eta_c$ (m) |
|---|---|---|---|---|---|---|
| 2 | 1.34 | 17.9 | 3.00 | 7.70 | 7.64 | 0.58 |

Table 3.15 – A height of dam crest (building breakwater wall)

| Phase | Water level (m) | $\Delta h$ (m) | $\eta_c$ (m) | $h_1$ (m) | a (m) | The top of wall (m) |
|---|---|---|---|---|---|---|
| 2 | 7.3 | 0.11 | 0.58 | 1.50 | 0.50 | 10.00 |

Note: To ensure aesthetic character of works, the break water wall is about 1.2 m high and combined as a handrail. Computing the dam crest elevation with breakwater wall resulting : $\eta$ c = 0.58m; elevation of the wall crest $\nabla$ wall crest= 13.20m; elevation of the dam top $\nabla$dam crest = 12.00m.

***Selection of the Dam Crest's Elevation***

It is proposed to build a 1.2m high breakwater wall on the dam crest surface. The wall top reaches to elevation $\nabla$wall=13.20m and the dam crest elevation is of $\nabla$dam crest = 12.00m.

***c) Dam Crest Width and Reinforcement***

Dam crest width is identified from the conditions of construction, management, traffic and arrangement of other works in the area.

Dam crest width also depends on the dam height and can be identified with the experiential formula: $B = 1.1*H^{0,5} + 1 = 1.1*7^{0,5} + 1 = 3.9m$

Accordinng to TCVN 8216: 2009, a width of a dam crest for the dam work of level IV is from 5 to 10m. It is proposed to select the with of the dam is 5m; the dam crest surface is reinforced with 20cm thick aggregate stone and asphaltic penetration of 5.5kg/m2.

***d) Dam Slopes Protection***

Dam slopes should be protected against the damages of rain, wave, seepage and other factors to ensure stability.

Downstream slope has generally not been impacted by waves and should be planted with grass for slope protection from erosion caused by rain run-off.

Upstream slope needs to be protected against wave. Some methods can be applied for the upstream slope protections are:

- Stone compacted linning
- Rip-rap
- Concrete slabs
- Precast Concrete pieces putting together.

Calculating a thickness of above protection layer under 14TCN 130-2002. Waving in the low water area for $H = 5.6m < \lambda/2 = 6m$ thereforce, a height of wave is:

$h_{1/3} = 1.53h_s = 1.53 \text{ x } 0.8 = 1.22m$

Using the Formula Pilarczyk to calculate a width of the reinforced layer: $\delta_B = \frac{h_{1/3}.\xi^{2/3}}{\varphi.\Delta}$

Table 3.16 – calculating widths of the reinforced layer

| Kind of reinforcement | δ (m) | $h_{1/3}$ | φ | Δ | ξ |
|---|---|---|---|---|---|
| Large size stone | 0.33 | 1.22 | 3 | 1.5 | 1.34 |
| riprap | 0.25 | 1.22 | 4 | 1.5 | 1.34 |
| Independent concrete piece | 0.21 | 1.22 | 5 | 1.4 | 1.34 |
| Interlock concrete block | 0.18 | 1.22 | 6 | 1.4 | 1.34 |

It is proposed to use 18cm sized interlock concrete blocks as they have high aesthetics.

### *3.4.3 Effluent flow Deep Culvert and flooding Spillway*

***a) Structure of Deep Oulet Culvert and Flooding Discharge Spillway.***

Due to some amount of water to be pumped into the reservoir, that the level of flooding overflow threshold should be designed to get the sufficient water depth for pump working as weel as not to cause flooding in the upstream area. As mentioned in the previous section, that the water level in the river bed may rise maximum to +4.0m will not cause the upstream areas flooding. To facilitate the pumping of water, the level of spillway threshold should be designed at +4.00 m. With the water level of +4.0m, the water depth in the river will reach 2.5m that is sufficient a condition on water column height for the pump working.

The structure of spillway is proposed to use the free-style overflow, level of the spill threshold +4.0m to ensure the water level for pumping and not to cause the flooding in upstream areas.

In addition, to drain sand and exhaust the river while the lake is full and not getting any water more, It is proposed to locate a Deep Oulet Culvert at the position of overflow route. The Deep Oulet Culvert is also tasked as the construction diversion during the time of the spillway execution. The deep drainage conduit structured by RC box culvert equiped with flat gate valve operated by electric motors.

***b) Dimension of the Deep Drainage Box Culvert and Splillway***

*Dimension of the Deep Drainage Box Culvert.*

The sewers are designed to drain sand and mud in the river bed, drain the regularly water flows in the river when the lake full and water diversion during the works construction.

To discharge sediments in the river bed, it needs to create a base current strong enough to flush sediments away.

And more, the box culvert size must ensure to diverse to water flow in dry season during the construction time. Results of hydraulic calculation with construction floods the dry months presented in Table 3.17 shows the construction water level in different periodically floods during the dry season. For construction water diversion, It requires not to cause water overflowing the river banks at elevation of +8.00 m, causing the flow back to the river bed to destroy river banks and surrounding dike. With such aboved conditions, the drain size of 1x (2,5 x2, 5)m has just carried the flood discharge in January; the Drain size of 2x (2,5 x2, 5)m can escape the flood flows in January to March and the Drain size of 3x (2 , 5x2, 5)m can escape the flood discharges of months from December to May next year. It is estimated the spillway construction time of about 5 months including 3 months to build a surrounding dike for drying up a spillway foundation pit area then It is proposed to choose the size of deep drainage box culvert of 2x (2,5 x2, 5) m. The darainage gate width of 5 meters equal to 2/3 width of the riverbed, fairly consistent and can generate base flows in the river bed to scour sand. To determine the ability to scour the river bed, It needs hydraulic model experiments in the next phase

Table 3.17 – The works execution flooding water level

| No | Q | (2.5x2.5)m | | 2x(2.5x2.5)m | | 3x(2.5x2.5)m | | Calculation Period |
|---|---|---|---|---|---|---|---|---|
| | ($m^3$/s) | $Z_{th}$(m) | $Z_h$(m) | $Z_{th}$(m) | $Z_h$(m) | $Z_{th}$(m) | $Z_h$(m) | |
| 1 | 34 | 5.74 | 4.28 | 4.92 | 4.28 | 4.55 | 4.28 | January |
| 2 | 52 | 8.10 | 4.87 | 6.07 | 4.87 | 5.55 | 4.87 | Feb. |
| 3 | 62 | 8.16 | 5.15 | 6.59 | 5.15 | 6.08 | 5.15 | Jan-Feb. |
| 4 | 88 | 8.27 | 5.74 | 8.00 | 5.74 | 7.25 | 5.74 | Jan-March |
| 5 | 104 | 8.34 | 6.05 | 8.18 | 6.05 | 7.92 | 6.05 | Dec- March |
| 6 | 121 | 8.41 | 6.34 | 8.29 | 6.34 | 8.16 | 6.34 | Dec.- April |
| 7 | 162 | 8.56 | 6.97 | 8.48 | 6.97 | 8.40 | 6.97 | Nov.- April |

*Dimension of flooding discharge spillway*

The spillway is designed to relieve all flooding discharges and not raise water level so much in compared to present water level. Using software of HEC-RAS to calculate water level at overflow weir giving the following results :

Table 3.18 – Up and downstream of Spillway water level corresponding to different sizes of spillway

| No | Frequency | Q | Existing | | B = 35m | | B = 30m | | B = 25m | | B = 20m | | B = 15m | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ($m^3/s$) | $Z_{th}$(m) | $Z_h$(m) | $Z_{th}$(m) | $Z_h$(m) | $Z_{th}$(m) | $Z_h$(m) | $Z_{th}$(m) | $Z_h$(m) | $Z_{th}$(m) | $Z_h$(m) | $Z_{th}$(m) | $Z_h$(m) |
| 1 | 0.20% | 1485 | 11.59 | 11.43 | 11.57 | 11.34 | 11.57 | 11.34 | 11.57 | 11.34 | 11.57 | 11.34 | 11.57 | 11.34 |
| 2 | 0.50% | 1285 | 11.16 | 11.00 | 11.14 | 10.90 | 11.14 | 10.90 | 11.14 | 10.90 | 11.14 | 10.90 | 11.14 | 10.9 |
| 3 | 1.00% | 1132 | 10.80 | 10.64 | 10.79 | 10.55 | 10.79 | 10.55 | 10.80 | 10.55 | 10.80 | 10.55 | 10.8 | 10.55 |
| 4 | 1.50% | 1083 | 10.69 | 10.52 | 10.68 | 10.44 | 10.68 | 10.44 | 10.69 | 10.44 | 10.69 | 10.44 | 10.69 | 10.44 |
| 5 | 5.00% | 784 | 9.91 | 9.73 | 9.94 | 9.66 | 9.94 | 9.66 | 9.95 | 9.66 | 9.96 | 9.66 | 9.96 | 9.66 |
| 6 | 10.0% | 628 | 9.45 | 9.26 | 9.51 | 9.20 | 9.51 | 9.20 | 9.52 | 9.20 | 9.53 | 9.20 | 9.54 | 9.2 |
| 7 | | 300 | 8.18 | 7.99 | 8.35 | 7.96 | 8.36 | 7.96 | 8.41 | 7.96 | 8.46 | 7.96 | 8.53 | 7.96 |
| 8 | | 200 | 7.52 | 7.39 | 7.70 | 7.39 | 7.71 | 7.39 | 7.79 | 7.39 | 7.93 | 7.39 | 8.13 | 7.39 |
| 9 | | 100 | 6.10 | 5.97 | 6.21 | 5.97 | 6.21 | 5.97 | 6.28 | 5.97 | 6.41 | 5.97 | 6.6 | 5.97 |

From the results of water level calculation it can get the following assessments:

- At flow of Q = 300 $m^3/s$ the water current overflow the river banks causing flooding in both sides of the river .
- With lagre flows of Q > 300 $m^3/s$, the influence of spillway gate to water level gradually reduces because if the capacity of flows increase, the run-off primarily flows on the bank ground surface. Corresponding to designed flood of Q = 1083 m3 / s, the spillway gate size (from 15m to 35m) hardly affect to water levels in rivers. Thus, based on flow and the designed flood level, there is no basis to select the appropriate spillway gate size.

According to the requirements to obtain water in gravity flow of phase 1, the spillway is designed overflow crest at elevation +4.00 m and the area in the river to drain flood is down about 23m2. To ensure conditions to drain flood discharge with little flows and the arrangement spillway is consistent with cross-sectional width of streams, It is proposed to select the spillway width B = 30m.

This spillway width is equal to existing cross sectional width of the river. With this size at the spill location, while the runoff flows in the river bed (Q <300m3 / s), the upstream overflow water level is higher than the current case of about 20cm; and it only occurs in a narrow area near the spill position and does not affect the upstream additional flooding caused by the construction of spillway.

### *3.4.5 Water Intake Pumping Station and Environmental maintained discharged Sewer*

#### *a) Structuure of Water Intake Pumping Station*

It has responsibilty to take water into the reservoirand the pump building is made of reinforced concrete. To restrict sediments flowing to the pump sump, the suction canal is proposedly designed at elevation of +2.0m, 0.5m higher the canal invert level and the invert level of pump sump will depend on the type of selected pumps. Bottom level of discharged water tank is same as the bottom level of the reservoir to spend out water enery and avoid causing erosion for the reservoir bed.

In addition, It is proposed to locate some $\phi$40 sewers with their invert level as same as the reservoir bottom level in order to discharged additional flows back to the river to maintain the environmental flow while the natural flow of the river is lower than 0.237 $m^3/s$.

***b) Selection of Type and number of Pumps***

The Pumping station should meet the requirement of taking water to the reservoir in different conditions. According to the water balance calculation, the largest inlet water flow in July is Q = 3.16 m3 / s, equal to the inlet water flow (3.41 m3 / s) minus the discharge for environment maintenance flow (0.24m3 / s). However, taking into account for the uneven distribution of the water current in months as well as monthly average flows between years, the intake water capacity should be higher than the average monthly flow. Empirically, the difference between the maximum average daily flow and the average monthly flow in the dry season or in early rainy season ranges from 1.5-2.5 times. The intake water capacity should also be taken to proportionally increase in those days that the natural water flows having large amount of water. Because there is no document on flow distribution in the months, It is proposed to get distribution coefficient for the flow fluctuation of 1.5. (Because in the rainy months, It can get more addional quantity of water that why the smaller value should be chosen to reduce a quantity of water required to be pumped while ensuring safety factor for water taking). Thus, the designed capacity of intake pumping station needs to be Q = 4.27 m3 / s to get enough water in the days of heavy natural water traffic.

In the rainy season months, the pumping capacity is equal to the water demand. For example, in Ocrtober, with the required flow of Qre = 1.13 $m^3/s$, the pumping capacity is equal the required flow minus the flow discharged for environmental maintenance $Q_2$=0.89 $m^3/s$. Considering to a fluctuation factor of flow distribution of 1.5 the water capacity to be pumped is 1.34.

In dry season months, in some months (such as in Febrary, March, April), the natural flows are small enough to discharge for environmental current maintenance; Therefore, the pumped water flows to the reservoir are too small and to be consider as 0.

Based on the monthly pumping capacity calculation, It is proposed to select capacity of pumps as follows:

- 3 Pumps with a capacity of Q=1.4 $m^3/s$.
- 2 Pumps with a capacity of Q=0.4 $m^3/s$.
- 1 Pumps with a capacity of Q=0.2 $m^3/s$.

With pump capacity as selected above, the water flows can be pumped to fit the calculated intake water flow in most months round a year. In February, the natural flow is very small and water can be dicontinuosly pumed every two days. In March of exhausted years if the natural flow in the river are too small, it will require to discharge water from the lake to the river for maitaining the environmental flow.

Because the intermittent pumping operation and the quatity of water to be pumped in July can partly move to be pumped in August, September or October, it will not require the stanby pumps. The time that pumps are not working can be used to rotate pump maintenance. In the case of a pump to be broken in the working time with the maximum pumping capacity (in July), a part of water quantity can be switchedly pumped in the August, September or October in order that the replacement or maintenance of pump failure can be done. The total pumped cacity is 4.75m3 /s.

Pump type: the submersible motor pumps are proposed to use.

A level of discharged conduit end : the end of discharge pipe is located 0.5m higher the normal water level of the reservoir and therefore the elevation of discharged pipe end is +11.00m

Pump head: Pump head is the height from the elevation of discharge pipe end to the water level in the river of elevation of +4.00m. Pump head H=7.0m

Table 3.19 – calculation of water flow to be pumped in months

**B?NG TÍNH L?U L??NG B?M**

| No | Month | Qinflow ($m^3$ s) | Weffluent ($10^6$ $m^3$) | Q redundant ($m^3$ s) | Qdischarge for environment ($m^3$ s) | Q intake ($m^3$ s) | fluctuation factor | Qpumped ($m^3$ s) |
|---|---|---|---|---|---|---|---|---|
| 1 | I | 0,48 | | | 0,237 | 0,24 | 1,50 | 0,36 |
| 2 | II | 0,26 | | | 0,237 | 0,02 | 1,50 | 0,03 |
| 3 | III | 0,20 | | | 0,237 | (0,04) | 1,50 | (0,05) |
| 4 | IV | 0,36 | | | 0,237 | 0,12 | 1,50 | 0,19 |
| 5 | V | 0,72 | | | 0,237 | 0,48 | 1,50 | 0,72 |
| 6 | VI | 1,63 | | | 0,237 | 1,39 | 1,50 | 2,09 |
| 7 | VII | 3,41 | | | 0,237 | 3,17 | 1,50 | 4,75 |
| 8 | VIII | 4,35 | 7,81 | 2,92 | 0,237 | 1,19 | 1,50 | 1,79 |
| 9 | IX | 5,58 | 11,50 | 4,44 | 0,237 | 0,90 | 1,50 | 1,35 |
| 10 | X | 6,87 | 15,37 | 5,74 | 0,237 | 0,89 | 1,50 | 1,34 |
| 11 | XI | 2,72 | 4,07 | 1,57 | 0,237 | 0,92 | 1,50 | 1,38 |
| 12 | XII | 1,04 | | | 0,237 | 0,81 | 1,50 | 1,21 |

Table 3.20 – Specifications of selected pumps

| No | Specifications | Value | | |
|---|---|---|---|---|
| 1 | Pump | PL 7065/735 | LL 3300 LT | NL 3300 LT |
| 2 | Number of pump | 3 | 2 | 1 |
| 3 | Pump head | 8 | 8 | 8 |
| 4 | Pump capacity (l/s) | 1375 | 340 | 165 |
| 5 | The maximum performance (%) | 79.5 | 77.8 | 78.6 |
| 6 | the wheel Code | 640 | 801 | 821 |
| 7 | Motor power | 140 | 37 | 27 |
| 8 | Oulet shaft power (Kw) | 136.9 | 37 | 21 |
| 9 | Power of inlet shaft (Kw) | 147.1 | 35 | 18 |
| 10 | Rpm (r/m) | 980 | 725 | 725 |
| 11 | 3-phase voltage | 380 | 380 | 380 |

Total capacity of the Pumping station: 4.97 $m^3$/s.

# CHAPTER 4 MEASURES FOR WORKS CONSTRUCTION EXECUTION

## 4.1 The conditions for the provision of materials, equipments, raw materials energy, infrastructure services

### *4.1.1 Situation of construction materials*

*A. Stone materials:*

In Phu Quoc, the stone-pits are not allowed to be exploited so all kinds of stone materials used for construction are transported from the mainland to the island. Stone can be obtained from Ha Tien, 70km seaway transported to Da Chong port of Phu Quoc island, then be transported to site by road distance of 15km

*B. Sand Materials:*

There is no survey on sand material in the project area. However, survey the area around the works site and consult local people that It can mine sand materials along the Cua can river for casting the concrete for the project. The average transportation distance of sand material is around 5km.

*C. Water Supply*

It is able to get water for construction from Cua Can stream. Water used for domestic activities suppiled from underground water wells or digged wells at the site.

*D. Earth filling*

A survey for spared soil in an area of 2.5ha in the reservoir area showed that volume of the CL soil layer can be calculated as follows: area of : spared soil area of 2.5 ha, average mined height of 1.9m. So the exploitable reverses = 25000m$^2$ x 1.9m = 47,500m$^3$

Average thickness of the CL soil layer should be exploited with 1m thick left below the layer bottom to keep the 1m soil layer against seepage over the lake bottom area (if the whole thickness of CL layer has been removed, the belowed soil layer of SC born a powerful easy-permeability appeares and causes the seepage flow through the reservoir bottom). According to the strata, CL layer wildly spreads in the lake area, so it is possible to expand the spared soil beach to exploit sufficient quantity of soil materials for earth filling to prevent seepage for the embankment construction.

### *4.1.2 Conditions for supplying Material, Equipments, Raw Materials*

The majority of construction materials such as iron, steel, cement, brick, stone, etc. .. will be provided from the nearest town of Ha Tien, 70 km sea way transportation to Da Chong sea port, then be transported to site by land road distance of 15km.

### *4.1.3 Enery Supply Conditions*

The Enery supplying for the construction services is maijority of electriccity power for construction activities, fuel for soil work machines. In the project area there is no existing national grid; However, during the time of the project construction, it may get the construction of the grid for the whole Phu Quoc Island and so it can take advantage of this grid to serve the worksconstruction execution.

Besides, the supply of electric power for construction execution will also be provided by the diesel generator with an appropriate capacity. That the power supply of construction such as this alternative ensures rationality on economic - technology.

### *4.1.4 Conditions for Providing infrasture Services*

a) Traffic: the project works, 20km far Duong Dong town, located next the traffic road connecting the South to the North of Phu Quoc island has been asphalt paved and. In such a position the conditions on traffic - transportation, supply of energy and materials for the construction is relatively favorable.

b) Communications Information: in the project area now there has the wired and wireless communications network. Mobile waves are relatively good. High-speed Internet service only developes in Duong Dong town, 20 km far from to the project works.

## 4.2 Analysis of Preliminary Selection of Construction Solutions

### *4.2.1 Measure for main works construction*

*1. <u>Solution for water diversion for works construction execution</u>*

*a) Designed Frequency*

Water Diversion: the Works level IV, p = 10 %

Blocking water current: the Works level IV, p = 10 %

*b) Hydrolgical documents in the construction time correstponding to frequency of P = 10%*

- Flooding Peak flows of the exhausted season months are calculated following methods similar to floods in the main flooding season. The results calculated shows in the following table.

Table 4.1: the Periodically construction Flooding peak Flow

*(Unit Qp: $m^3/s$)*

| No | Periodically Flooding Peak Flow (P =10%) | | | | | |
|---|---|---|---|---|---|---|
| | XII-IV | XII-III | I - III | I - II | I | II |
| 1 | 121,0 | 104,1 | 87,7 | 62,6 | 33,8 | 52,2 |

*c) Solutions Applied for Water Current Diversion and the Works Construction Execution*

Based on the mentioned hydrological documents, volume of works and capacity of work execution and through a comparison of the advantages and disadvantages of the periodical flow diversion during the construction time, it can recognize that the selection time for construction water diversion in dry season should be from January to March. The frequency of construction water diversion in the dry season of P = 10% with a diversion flow of QK = 87.7 m3 / s. in Flooding season, frequency of flood water diversion for P = 10% with a diversion flow of QL = 628 m3 / s.

Because the reservoir area is independent from the river bed, thereforce, the Reservoir construction execution do not require special construction methods applied for the construction water diversion and it has just required to dig the small drainage ditches during construction time. Flow Diversion measures will be implemeted during the construction of spillway and deep oulet culvert. Based on the topographical conditions and works structures, It is proposed that the water flow in dry season will be diverted through the deep oulet culvert and discharges through the spillway or river in the flood season.

The Deep Oulet of RC Box culvert has dimensions of 2x(2.5mx2,5m) and invert level of ∇+1.50m.

Based on the charaters of the works structures and quantity of work to be executed, It is proposed the duration of works execution is 3 years for the whole project. The order of water diversion for the works construction as follows:

+ The frist year: Ecavating soil in a haft area of the reservoir, constructing the earth-damp, earth-filling on the reservoir bottom area for against seepage. Water flows in the natural river.

+ The Second Year: Excavating a remain haft area of the reservoir, constructing earth-damp, earth-filling on the reservoir bottom area for against seepage. Water flows in the natural river.

+ The third year: constructing the intake sluice, deep oulet culvert in the dry season from December to January next year and during the construction time, the water flows in the natural river; From January to Marth, executing the construction of spillway and flows divertes through the deep oulet culvert with dimensions of 2x(2.5x2.5)m. From the Marth to December: It is completed the works and the water diverted over the spills and the deep oulet culvert.

*d) Works of Blocking the Water Current*

The flows of blocking current is calculated for the months having minimum flows in the exhausted season. As expected in the construction schedule, in the early of April, the water traffic line will be blocked, so the current blocked flows get a frequency of 10% of Q = 1.7 m3/s. As in the dry season, water flows are diversed through the deep outlet culvert, the block of the water current is simply by dropping the block fades after the construction of surrounding dike is complete.

*2. Technical Solutions for The Main Works Construction*

*(i) Soil Excavation – earth filling to build a reservoir.*

Works of soil excavation, earth filling to creature the reservoir include:

- Excavating the soil in the reservoir area to bottom designed level;
- Earth filling to create a soil layer against seepage for the reservoir bottom and bank slopes;
- Dam up surrounding the reservoir;
- Reinforcing up and downstream of the reservoir slopes.

In order to save the cost of construction, soil excavation and transportation, the above-mentioned earthworks should be done concurrently in the form of rolling. The reservoir bed is divided into many different areas, each is made by different stage and getting excavated soil which has high capacity of seepage prevention in one area filling up for the another adjacent area to reduce the soil transportation. After completion of the earthworks, the new earth dam slopes should be reinforced to avoid motors and vehicles served for works execution to cause damage for the earth dam's slopes. The Earthworks to dam up needs to be done in the dry season to ensure the quality of earth-filling.

The layers of soil in the reservoir area as clay or sandy-clay, dense, stiff, equivalent to the soil grade III; the soil in the reservoir area to be dug by excavators of 2.3 m3 capacity bucket, dumped up cars of 12 tons loading and transported to allowable area.

Due to huge volume of excavated soil, at present it has not yet found the most appropriate solution for loading the huge quantity of unwanted soil. In this design, Excavated soil are expected to be concentratedly loaded on temporary dump in the lake bed and used for other projects in the island along the sea coast to raise their ground level. In the case, if it is does not combine the excavation of soil with using the unwanted quantity of spoil for other projects ground levelling, it should contact with local authorities to find out an appropriate spoil loading area.

*(ii) A Drainage of storm water for The Foundation Pit.*

The construction execution of the reservoir and other works is majorily done in the dry season, so the river bed is almost no water. Except for the work of digging ditches to drain water in the lake, it requires pumping the water from foundation pit caused by seepage flow, or stagnant water in the lake during construction time. It is estimated to use 10 pumps with a capacity of 20CV for pumping water during construction time.

*(iii) Concrete Construction.*

Due to quantity of concrete is not large that the whole quantity of concrete will be mixed at site by the mixer with a capacity 1000 liters to 2000 liters, then using the crane to pour concrete mortar into the plot.

#### *4.2.2 Organization of Construction*

*1. General layout of construction work*

*(i) The miscellaneous*

The plan is arranged into two distinct areas. Production zone covers the material dump, workshop, concrete mixing station, motorcycles leaving, petrol storage, etc. … other area for domestic activies include manager board, working area, residential area of employee and workers, cultural activities, sport parks, the postal service, retaurants, food, groceries. The layou for miscellaneous works is arranged in the lake area to reduce required area and also take advantage of flood control by dam around the lake. In the last year of the construction time, It can move these works to outside the reservoir area to comletedly construct the reservoir.

A part of the reservoir bed has been arranged as the temporarily spoil dump before the it can be transported to fill the ground for other areas.

Total area of the temporarily spoil dump , miscellaneous area and house serve for construction work is estimated as around : **50.000 m$^2$**.

*2. Total Construction Progress*

*(i) The requirements and the basis for setting up the construction progress*

The construction progress is established based on the followings:

- A period of the work execution has been suggested as 3 years
- Measure of flow diversion for constructing works: water flows diversed through the river or the deep outlet culvert.
- Quantity and features of the project works.

*(ii) Construction progress*

Table 4.2 – Total construction progress

| No. | items | the first year | | | | The second year | | | | the Third year | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | I - III | IV - VI | VII - IX | X - XII | I - III | IV - VI | VII - IX | X - XII | I - III | IV - VI | VII - IX | X - XII |
| 1 | Soil excanation work | | | | | | | | | | | | |
| 2 | Earth-filling work | | | | | | | | | | | | |
| 3 | Dam slopes reinforcement | | | | | | | | | | | | |
| 4 | Constructing deep outlet culvert - spillway | | | | | | | | | | | | |
| 5 | Construction of the Intake Sluice | | | | | | | | | | | | |
| 6 | Reinforcement of the Dam crest and completion of works | | | | | | | | | | | | |

# CHAPTER 5 QUANTITY OF WORKS & INVESTMENT CAPITAL

## 5.1 Quantity of works & Construction Cost

Table 5.1 – Detailed Cost Estimation of Alternative 1

| No | Work | Unit | Quantity | Unit price(Dong) | Amount of money (Dong) | Remark |
|---|---|---|---|---|---|---|
| **I** | **The Reservoir - earth dam** | | | | **339,012,195,000** | |
| 1 | soil excavation | $m^3$ | 10,271,215 | 20,000 | 205,424,306,000 | Transportation of 1km |
| 2 | Soil excavated for filling | $m^3$ | 2,273,785 | 20,000 | 45,475,694,000 | Transportation of 1km |
| 3 | earth-filling for against seepage of the reservoir bed | $m^3$ | 1,930,000 | 10,000 | 19,300,000,000 | |
| 4 | Dam earth-filling | $m^3$ | 137,077 | 20,000 | 2,741,540,000 | |
| 5 | Concrete grade 200 for dam slopes reinforced | $m^3$ | 12,388 | 1,500,000 | 18,582,000,000 | |
| 6 | C.grade200 of breakwater wall | $m^3$ | 4,907 | 1,700,000 | 8,341,900,000 | |
| 7 | Aggregate stone (1x2) upstream filter layer | $m^3$ | 8,334 | 600,000 | 5,000,400,000 | |
| 8 | Filtration clothe | $m^2$ | 83,342 | 40,000 | 3,333,680,000 | |
| 9 | Reinforced steel | kg | 888,000 | 23,000 | 20,424,000,000 | |
| 10 | Aggregate stone 0-4 for dam crest surface | $m^3$ | 4,992 | 550,000 | 2,745,600,000 | |
| 11 | Alphaltic infiltration of 5.5kg/$m^2$ for dam crest surface | $m^2$ | 24,960 | 200,000 | 4,992,000,000 | |
| 12 | Grass | $m^2$ | 29,741 | 15,000 | 446,115,000 | |
| 13 | Cashing | $m^2$ | 27,562 | 80,000 | 2,204,960,000 | |
| **II** | **Spillway and Deep outlet Culvert** | | | | **12,691,855,000** | |
| 1 | C. grade 200 | $m^3$ | 2,214 | 1,500,000 | 3,321,000,000 | |
| 2 | C. grade 300 | $m^3$ | 30 | 1,700,000 | 51,000,000 | |
| 3 | all kind of steel | kg | 173,000 | 23,000 | 3,979,000,000 | |
| 4 | Linning C. grade 100 | $m^3$ | 301 | 1,200,000 | 361,200,000 | |
| 5 | Aggregate stone (1x2) upstream filter layer | $m^3$ | 70 | 600,000 | 42,000,000 | |
| 6 | Filtration clothe | $m^2$ | 703 | 40,000 | 28,120,000 | |
| 7 | Cashing | $m^2$ | 2,631 | 80,000 | 210,480,000 | |
| 8 | Excavated soil | $m^3$ | 93,503 | 25,000 | 2,337,575,000 | Transportation of 1km |
| 9 | Earth - filling | $m^3$ | 1,537 | 40,000 | 61,480,000 | Transportation of 1km |
| 10 | Sluice gate Machenics | kg | 16,000 | 100,000 | 1,600,000,000 | |
| 11 | Electrical operation | Set | | | 700,000,000 | |
| **III** | **Intake sluice** | | | | **3,621,000,000** | |

| No | Work | Unit | Quantity | Unit price(Dong) | Amount of money (Dong) | Remark |
|---|---|---|---|---|---|---|
| 1 | C. grade 200 | $m^3$ | 588 | 1,500,000 | 882,000,000 | |
| 2 | C. grade 300 | $m^3$ | 1 | 1,700,000 | 1,700,000 | |
| 3 | all kind of steel | kg | 48,000 | 23,000 | 1,104,000,000 | |
| 4 | Linning C. grade 100 | $m^3$ | 22 | 1,200,000 | 26,400,000 | |
| 5 | Aggregate stone (1x2) upstream filter layer | $m^3$ | 35 | 600,000 | 21,000,000 | |
| 6 | Filtration clothe | $m^2$ | 344 | 40,000 | 13,760,000 | |
| 7 | Cashing | $m^2$ | 767 | 80,000 | 61,360,000 | |
| 8 | Excavated soil | $m^3$ | 4,442 | 30,000 | 133,260,000 | Transportation of 1km |
| 9 | Earth - filling | $m^3$ | 1,938 | 40,000 | 77,520,000 | Transportation of 1km |
| 10 | Intake gate Machenics | kg | 8,000 | 100,000 | 800,000,000 | |
| 11 | Electrical control | set | | | 500,000,000 | |
| **IV** | **Raw water Intake pumping station** | | | | **27,472,406,000** | |
| 1 | Concrete grade 250 | $m^3$ | 778 | 1,500,000 | 1,167,000,000 | |
| 3 | all kind of steel | kg | 92,842 | 23,000 | 2,135,366,000 | |
| 4 | Concrete piles | m | 1,440 | 900,000 | 1,296,000,000 | |
| 5 | Linning C. grade 100 | $m^3$ | 42 | 1,200,000 | 50,400,000 | |
| 6 | Formwork | $m^2$ | 2,483 | 80,000 | 198,640,000 | |
| 7 | Đất đào | $m^3$ | 2,000 | 30,000 | 60,000,000 | |
| 8 | Excavated soil | $m^3$ | 1,000 | 40,000 | 40,000,000 | |
| 9 | PVC joints | m | 50 | 500,000 | 25,000,000 | |
| 10 | Pumping pipe | kg | 10,000 | 100,000 | 1,000,000,000 | |
| 11 | Electrical control | set | | | 500,000,000 | |
| 12 | Pum PL 7065/735 | cái | 3 | | 12,600,000,000 | |
| 13 | Pump LL 3300 LT | cái | 2 | | 6,300,000,000 | |
| 14 | Pump NL 3300 LT | cái | 1 | | 2,100,000,000 | |
| **V** | **Preparation of works plan (5%)** | | | | **19,742,413,800** | |
| | **Total** | | | | **414,590,689,800** | |

Table 5.2 – General total construction cost – Alternative 1

| No. | Items | Amount of Money (dong) | Remark |
|---|---|---|---|
| **1** | **Construction cost before tax (rounded)** | **414,590,689,800** | |
| 2 | VAT | 41,459,068,980 | |
| **3** | **Construction Cost after tax** | **456,049,758,780** | |
| 4 | Quantiyof work contingency (10%) | 45,604,975,878 | |
| **5** | Contingency for Slippage in price (20%) | 91,209,951,756.00 | |
| 6 | **Total construction cost (rounded)** | **592,864,686,000** | |

Table 5.3 – Detailed Cost Estimation of Alternative 2 ( selected)

| No. | Work | Unit | Quantity | Unit price (dong) | Amount of Money(dong) | Remark |
|---|---|---|---|---|---|---|
| **I** | **The reservoir- Earth Dam** | | | | **217,866,525,000** | |
| 1 | Excavated soil | $m^3$ | 5,351,919 | 20,000 | 107,038,382,000 | Transportation of 1km |
| 2 | Soil excavated for filling | $m^3$ | 1,017,081 | 20,000 | 20,341,618,000 | Transportation of 1km |
| 3 | Earth-filling for against seepage of the reservoir bed | $m^3$ | 386,000 | 10,000 | 3,860,000,000 | |
| 4 | Earth dam filling | m3 | 538,619 | 20,000 | 10,772,380,000 | |
| 5 | Concrete grade 200 for dam slopes reinforced | m3 | 12,388 | 1,500,000 | 18,582,000,000 | |
| 6 | C.grade 200 of breakwater wall | $m^3$ | 4,907 | 1,700,000 | 8,341,900,000 | |
| 7 | Aggregate stone (1x2) upstream filter layer | m3 | 11,584 | 600,000 | 6,950,400,000 | |
| 8 | Filtration clothe | $m^2$ | 115,842 | 40,000 | 4,633,680,000 | |
| 9 | Reinforced steel | kg | 888,000 | 23,000 | 20,424,000,000 | |
| 10 | Aggregate stone 0-4 for dam crest surface | $m^3$ | 4,992 | 550,000 | 2,745,600,000 | |
| 11 | Alphaltic penetration of 5.5kg/$m^2$ for dam crest surface | m2 | 24,960 | 200,000 | 4,992,000,000 | |
| 12 | Grassing | $m^2$ | 75,307 | 15,000 | 1,129,605,000 | |
| 13 | Formwork | $m^2$ | 27,562 | 80,000 | 2,204,960,000 | |
| 14 | Construction stone | $m^3$ | 9,750 | 600,000 | 5,850,000,000 | |
| **II** | **Spillway and Deep outlet Culvert** | | | | **17,462,675,000** | |
| **III** | **Pumping station** | | | | **29,757,310,800** | |
| 1 | C. grade 250 | $m^3$ | 934 | 1,500,000 | 1,400,400,000 | |
| 2 | All kind of steel | kg | 120,695 | 23,000 | 2,775,975,800 | |
| 3 | C.piles | m | 1,440 | 900,000 | 1,296,000,000 | |
| 4 | Lining Concrete grade 100 | $m^3$ | 46 | 600,000 | 27,720,000 | |

| No. | Work | Unit | Quantity | Unit price (dong) | Amount of Money(dong) | Remark |
|---|---|---|---|---|---|---|
| 5 | Formwork | $m^2$ | 2,731 | 550,000 | 1,502,215,000 | |
| 6 | Excavated soil | $m^3$ | 5,000 | 30,000 | 150,000,000 | |
| 7 | Filled soil | $m^3$ | 2,000 | 40,000 | 80,000,000 | |
| 8 | PVC joint | m | 50 | 500,000 | 25,000,000 | |
| 9 | Pumping pipes | kg | 10,000 | 100,000 | 1,000,000,000 | |
| 10 | Electrical control | set | | | 500,000,000 | |
| 11 | Pump PL 7065/735 | Piece | 3 | | 12,600,000,000 | |
| 12 | Pump LL 3300 LT | Piece | 2 | | 6,300,000,000 | |
| 13 | Pump NL 3300 LT | Piece | 1 | | 2,100,000,000 | |
| **IV** | **Preparing the works plan (5%)** | | | | **13,254,325,540** | |
| | **TOTAL** | | | | **278,340,836,340** | |

Table 5.4 – General total construction cost – Alternative 2

| No. | Items | Amount of Money (dong) | Remark |
|---|---|---|---|
| **1** | **Construction cost before tax (rounded)** | **278,341,000,000** | |
| 2 | VAT | 27,834,100,000 | |
| **3** | **Construction Cost after tax** | **306,175,100,000** | |
| 4 | Quantiyof work contingency (10%) | 30,617,510,000 | |
| **5** | Contingency for Slippage in price (20%) | 61,235,020,000.00 | |
| 6 | **Total construction cost (rounded)** | **398,027,630,000** | |

Table 5.5 – Detailed Cost Estimation of Alternative 3

| No. | Work | Unit | Quantity | Unit price (dong) | Amount of Money(dong) | Remark |
|---|---|---|---|---|---|---|
| **I** | **The Reservoir- Earth Dam** | | | | **148,867,095,000** | |
| 1 | Excavated soil | $m^3$ | 1,065,181 | 20,000 | 21,303,614,000 | Transportation of 1km |
| 2 | Soil excavated for filling | $m^3$ | 1,443,819 | 20,000 | 28,876,386,000 | Transportation of 1km |
| 3 | Earth-filling for against seepage of the reservoir bed | $m^3$ | 386,000 | 10,000 | 3,860,000,000 | |
| 4 | Earth dam filling | $m^3$ | 926,563 | 20,000 | 18,531,260,000 | |
| 5 | Concrete grade 200 for dam slopes reinforced | $m^3$ | 12,388 | 1,500,000 | 18,582,000,000 | |
| 6 | C.grade200 of breakwater wall | $m^3$ | 4,907 | 1,700,000 | 8,341,900,000 | |
| 7 | Aggregate stone (1x2) upstream filter layer | $m^3$ | 11,584 | 600,000 | 6,950,400,000 | |
| 8 | Filtration clothe | $m^2$ | 115,842 | 40,000 | 4,633,680,000 | |
| 9 | Reinforced steel | kg | 888,000 | 23,000 | 20,424,000,000 | |
| 10 | Aggregate stone 0-4 for dam crest surface | $m^3$ | 4,992 | 550,000 | 2,745,600,000 | |

| No. | Work | Unit | Quantity | Unit price (dong) | Amount of Money(dong) | Remark |
|---|---|---|---|---|---|---|
| 11 | Alphaltic penetration of 5.5kg/m$^2$ for dam crest surface | m$^2$ | 24,960 | 200,000 | 4,992,000,000 | |
| 12 | Grassing | m$^2$ | 104,753 | 15,000 | 1,571,295,000 | |
| 13 | Formwork | m$^2$ | 27,562 | 80,000 | 2,204,960,000 | |
| 14 | Construction stone | m$^3$ | 9,750 | 600,000 | 5,850,000,000 | |
| **II** | **Spillway and Deep outlet Culvert** | | | | **17,462,675,000** | |
| **III** | **Pumping station** | | | | **29,757,310,800** | |
| **IV** | **Preparing the works plan (5%)** | | | | **9,804,354,040** | |
| | **Total** | | | | **205,891,434,840** | |

Table 5.6 – General total construction cost – Alternative 3

| No. | Items | Amount of Money (dong) | Remark |
|---|---|---|---|
| **1** | **Construction cost before tax (rounded)** | **205,891,435,000** | |
| 2 | VAT | 20,589,143,500 | |
| **3** | **Construction Cost after tax** | **226,480,578,500** | |
| 4 | Quantiyof work contingency (10%) | 22,648,057,850 | |
| **5** | Contingency for Slippage in price (20%) | 45,296,115,700.00 | |
| 6 | **Total construction cost (rounded)** | **294,424,752,000** | |

## 5.2 Arragement of unwanted soil and It influences to the work construction cost

Due to huge volume of excavated soil, at present it has not yet found the most appropriate solution for loading the huge quantity of unwanted soil. In this design, Excavated soil are expected to be concentratedly loaded on temporary dump in the lake bed and used for other projects in the island along the sea coast to raise their ground level. The average distance of soil transportation is around 1km.

In the case, if it is does not combine the excavation of soil with using the unwanted quantity of spoil for other projects ground levelling, the unwanted quantity of execavated soil will load on the fixed area and the soil transported distance will be further and the construction cost will increase. Volume of soil to be transported to the junk yard ( Alternative 2) is 5.3 millions m3. The following table will project the construction costs for the different transport distance:

Table 5.7 – the construction cost corresponding the differently soil transported distance

| No | Distance of soil Transportation | Quantity | Unit price | Cost of soil transportation | Construction cost before tax | Construction cost after tax x1.1 | Physical Contigency (10%) | Contigency for slipage in price (20%) | Total construction cost |
|---|---|---|---|---|---|---|---|---|---|
| | (km) | (m$^3$) | (dong) | (10$^6$ dong) | (10$^6$ dong) | (10$^6$ dong) | (10$^6$ dong) | (10$^6$ dong) | (10$^6$ dong) |
| 1 | 1 | 5,351,919 | 10,000 | 53,519.2 | 278,340.8 | 306,174.9 | 30,617.5 | 61,235.0 | 398,027.4 |
| 2 | 2 | 5,351,919 | 14,000 | 74,926.9 | 299,748.5 | 329,723.4 | 32,972.3 | 65,944.7 | 428,640.4 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 5 | 5,351,919 | 25,200 | 134,868.4 | 359,690.0 | 395,659.0 | 39,565.9 | 79,131.8 | 514,356.7 |
| 4 | 10 | 5,351,919 | 42,400 | 226,921.4 | 451,743.0 | 496,917.3 | 49,691.7 | 99,383.5 | 645,992.5 |
| 5 | 15 | 5,351,919 | 57,600 | 308,270.5 | 533,092.2 | 586,401.4 | 58,640.1 | 117,280.3 | 762,321.8 |
| 6 | 20 | 5,351,919 | 70,800 | 378,915.9 | 603,737.5 | 664,111.3 | 66,411.1 | 132,822.3 | 863,344.6 |

### 5.3 Investment Phasing

The project will be invested in two phases. In the pahse 1, the raw water will be supplied for WTP with a capacity of 21.000 $m^3$/day, by gravity intaken flow. Phase 2 the raw water will be supplied for WTP with a capacity of 52.500 $m^3$/day, by gravity intaken flow and pumping flow if the water level in the reservoir is higher than +4.00m. The investment of main works should be completed in the first stage.

# CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

## 6.1 Conclusions on The Project Work Construction Solutions

**The work Function:** Supplying raw water for the WTP with a capacity of phase 1 21,000 $m^3$/day and of phase 2 : 52,500 $m^3$/day; discharging water to maintain the environmental flow in dry season.

**Arrangement of the Reservoir Plan:** the plan of Cua can reservoir has been arranged; Cua Can reservoir is allocated towards a direction that it can keep the original status of Cua can river run-off; the reservoir will be built next to the river and the reservoir water is taken through the intake canal and pumps. The reservoir area is expanded to the Southwest of around 60ha in compared with the original layout.

**Scale of the reservoir is presented in the following table:**

The reservoir of alternative 2 has the following paramaters:

| No | The Reservoir Scale | Unit | Phase 1 | Phase 2 |
|---|---|---|---|---|
| 1 | The reservoir bottom level | m | 5,0 | 5,00 |
| 2 | Dead water level | m | 5,60 | 5,90 |
| 3 | Normal water level | m | 7,20 | 10,50 |
| 4 | Daed Volume: $W_c$ | $10^6$ $m^3$ | 1,13 | 1,69 |
| 5 | Total capacity: $W_{tp}$ | $10^6$ $m^3$ | 4,15 | 10,47 |
| 6 | Effective Capacity: $W_{hi}$ | $10^6$ $m^3$ | 3,02 | 8,77 |

**Seepage Prevention for the Water Reservoir:** With a designed elevation of the reservoir bottom at +5.0 m, most area of the reservoir bottom will lie on the highly unpermeable clay layer of CL or sandy-clay layer which prevent water loss from the lake and thereforce, it does not require applying the construction measures to prevent the seepage for the reservoir bottom. However, It is proposed to use the seepage prevention method for 20% of the reservoir bottom area with good waterproof clay soil layer of 1.0 m thick.

**Earth dam surrounding the reservoir:** soil filling for a dam up arrouding the reservoir execavated from the reservoir bed. The dam crest level is +12.0m mounted on with a break water wall reached to elevation of +13.2m. The dam crest surface is 5m wide, reinforeced with aggregate stone of 20cm thick and asphaltic penetration of 5.5kg/$m^2$. The upstream of the Dam slope is reinforced by interlocked concrete blocks of 18 cm thick and underneath with aggregate stone of 10cm thick and filter clothe layers.

**Floods discharge spillway :** is made of concrete, free overflow type, practical section with dimensions of 3x10m, elevation of spilltop of +4.10m, overflow designed discharge $Q_{1.5\%}$= 1083 $m^3$/s.

**Deep Oulet Culvert:** is a RC box culvert, equipped with falt gate valve operation by electrical motor. Dimesions of Box culvert are 2x(2.5x2.5)m, Invert level of +1.50m.

**The Raw water intake Pumping Station:** It is proposed to use the Submersible Motor Pumps; the pumping house is made of reinforced concrete, a Total capacity of the pumping station of 4.97 $m^3/s$; in the pumpingstation it locates $\phi$40 sewers to discharge the water for environmental flow maintenance.

The parameters of the pumping station as below

| No | Specifications | Value | | |
|---|---|---|---|---|
| 1 | Pumps | PL 7065/735 | LL 3300 LT | NL 3300 LT |
| 2 | Number of Pumps | 3 | 2 | 1 |
| 3 | Pump Head | 8 | 8 | 8 |
| 4 | Pump capacity (l/s) | 1375 | 340 | 165 |
| 5 | Largest performance (%) | 79.5 | 77.8 | 78.6 |
| 6 | The Wheel code | 640 | 801 | 821 |
| 7 | Motor power | 140 | 37 | 27 |
| 8 | Oulet shaft power (Kw) | 136.9 | 37 | 21 |
| 9 | Inlet shaft power (Kw) | 147.1 | 35 | 18 |
| 10 | Rpm (rounds/minutes) | 980 | 725 | 725 |
| 11 | 3-phases voltage | 380 | 380 | 380 |

**Construction progress:** Proposed 3 years.

**Total construction cost:.**

**Investment Phasing :** all main items will be invested in phase 1.

### 6.2 Recommendations on shortcomings that should be studied in the later stages.

***For Topography :***most topographical maps at a scale of 1/2,000 to 1/5,000 having limited accurateness unless the five surveyed cross-sections of the Cua can stream in different locations. Based on these available documents, the calculations for works design has just achieved the level of the preliminary stages; so in the next stage, It needs to survey the entire topography of the area, especially cross sections of the cua can river

***For Geology:*** Because the geological survey of this period is limited than in the following stages, It needs to execute additional surveys to elucidate strata of the reservoir area, particularly paying attention to the reservoir permeability and clay soil materials to prevent seepage.

It needs to execute additional ckecked and measured surveys on elementary hydrological factors such as: to elucidate stratigraphic reservoir area is particularly interested in water issues takes the reservoir permeability and materials waterproof clay

***For Hydrology :*** There is no actual measured documents of water current in the studied basin, So It needs to execute ckecked and measured surveys on elementary hydrological factors such as: regularly flows, flooding flows, sediments flows, quality of water at the works alignment to serve for calculation of the later stage. These factors are used as the actual basis for testing the parameters which have calculated and adjusting them if necessary.

***For Calculation of the Work Design***

- Because the measuring river cross-sections are limited, the calculation of stream flows may be inaccurate. In a later stage It needs to recalculate the stream flows based on the more accurately cross-sections of river documents.
- Collecting additional hydrologic documents, checking the calculations on the basis of the updated documents and contrasting comparedly to the vicinity projects which have been built or are building.

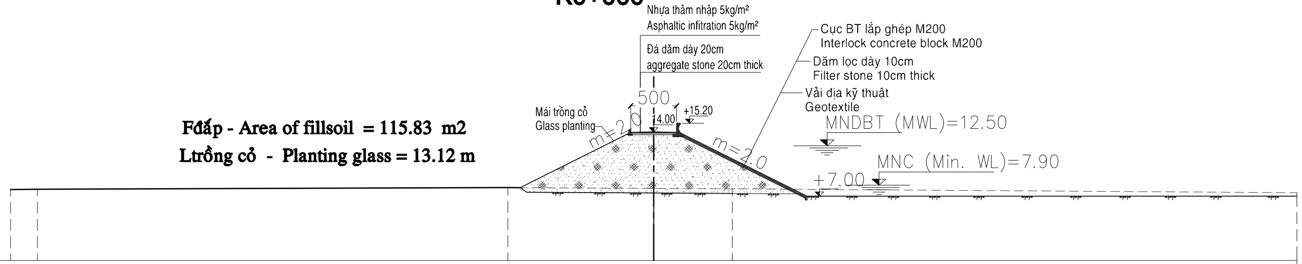
***For the excavated soil loading dump:*** Because the soil transportation costs greatly affect the works construction cost, thereforce, in the next stage, it should contact with local authorities to find out the spoil dump area nearst the lake to reduce the total construction costs. It can be combined with other projects required the soil for ground filling in order to get the plans for exploitation of soil in the reservoir bed, reducing the amount of soil excavation. In the case it cannot find out the spared area for loading unwanted soil, the alternative 3 should be considered to apply for the project implemetation.


TITLE :

RESERVOIR GENERAL LAYOUT PLAN

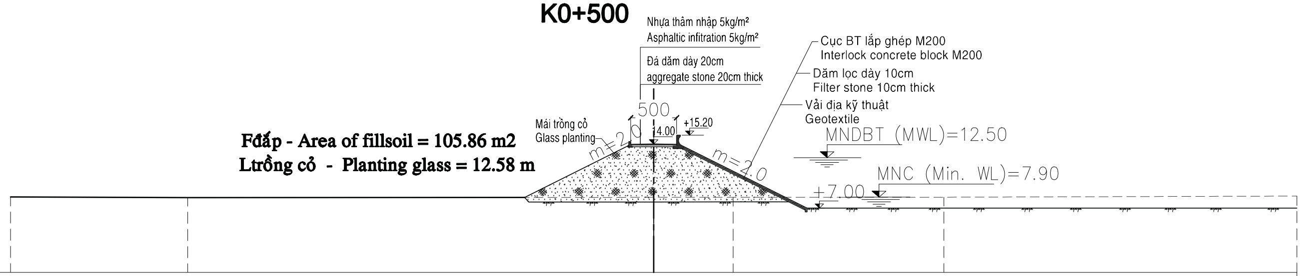
| PHU QUOC<br>Water Supply and Sewerage System Development<br>Preparatory Survey | Scale :<br>1:8000 | No :<br>R-1 |
|---|---|---|

## K0+000



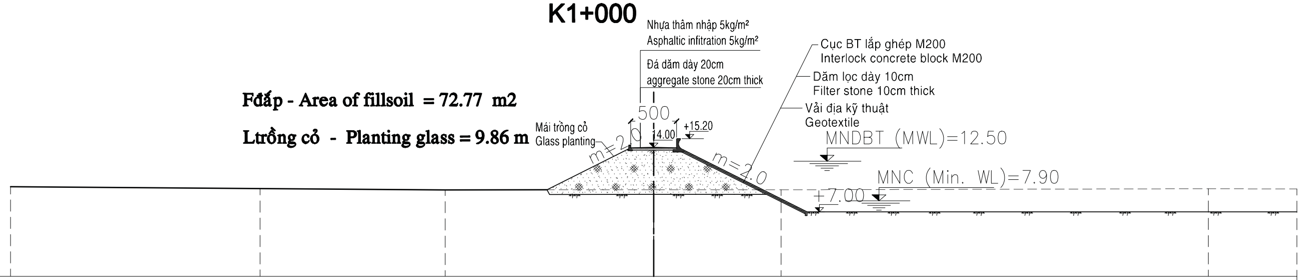

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Cao độ tự nhiên | 7.72 | | 7.75 | | 7.94 | | 7.85 | | 7.80 | | 7.38 |
| Khoảng cách mia | | 2.93 | | 51.20 | | 15.87 | | 8.56 | | 61.44 | |
| Cao độ thiết kế | 7.72 | | 7.75 | | 7.94 | | 7.85 | | 7.80 | | 7.38 |
| Kh/c lẻ thiết kế | | 2.93 | | 51.20 | | 15.87 | | 8.56 | | 61.44 | |

## K0+500




| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Cao độ tự nhiên | 8.22 | | 8.15 | | 8.18 | | 8.18 | | 8.19 | | 8.32 |
| Khoảng cách mia | | 19.29 | | 50.71 | | 8.65 | | 22.93 | | 38.42 | |
| Cao độ thiết kế | 8.22 | | 8.15 | | 8.18 | | 8.18 | | 8.19 | | 8.32 |
| Kh/c lẻ thiết kế | | 19.29 | | 50.71 | | 8.65 | | 22.93 | | 38.42 | |

## K1+000




| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Cao độ tự nhiên | 9.73 | | 9.53 | | 9.39 | | 9.39 | | 9.40 | | 9.51 | | 9.48 |
| Khoảng cách mia | | 27.15 | | 20.15 | | 22.69 | | 13.87 | | 46.49 | | 9.64 | |
| Cao độ thiết kế | 9.73 | | 9.53 | | 9.39 | | 9.39 | | 9.40 | | 9.51 | | 9.48 |
| Kh/c lẻ thiết kế | | 27.15 | | 20.15 | | 22.69 | | 13.87 | | 46.49 | | 9.64 | |



| | | |
|---|---|---|
| TITLE : RESERVOIR SECTION OF DAM (1) | | |
| PHU QUOC<br>Water Supply and Sewerage System Development<br>Preparatory Survey | Scale :<br>1:600 | No :<br>R-2 |