Jhimruk Khola Intake

DESIGN OF INTAKE AND DESILTING ARRANGEMENT FOR THE JHIMRUK HYDRO-ELECTRIC PROJECT (NEPAL)

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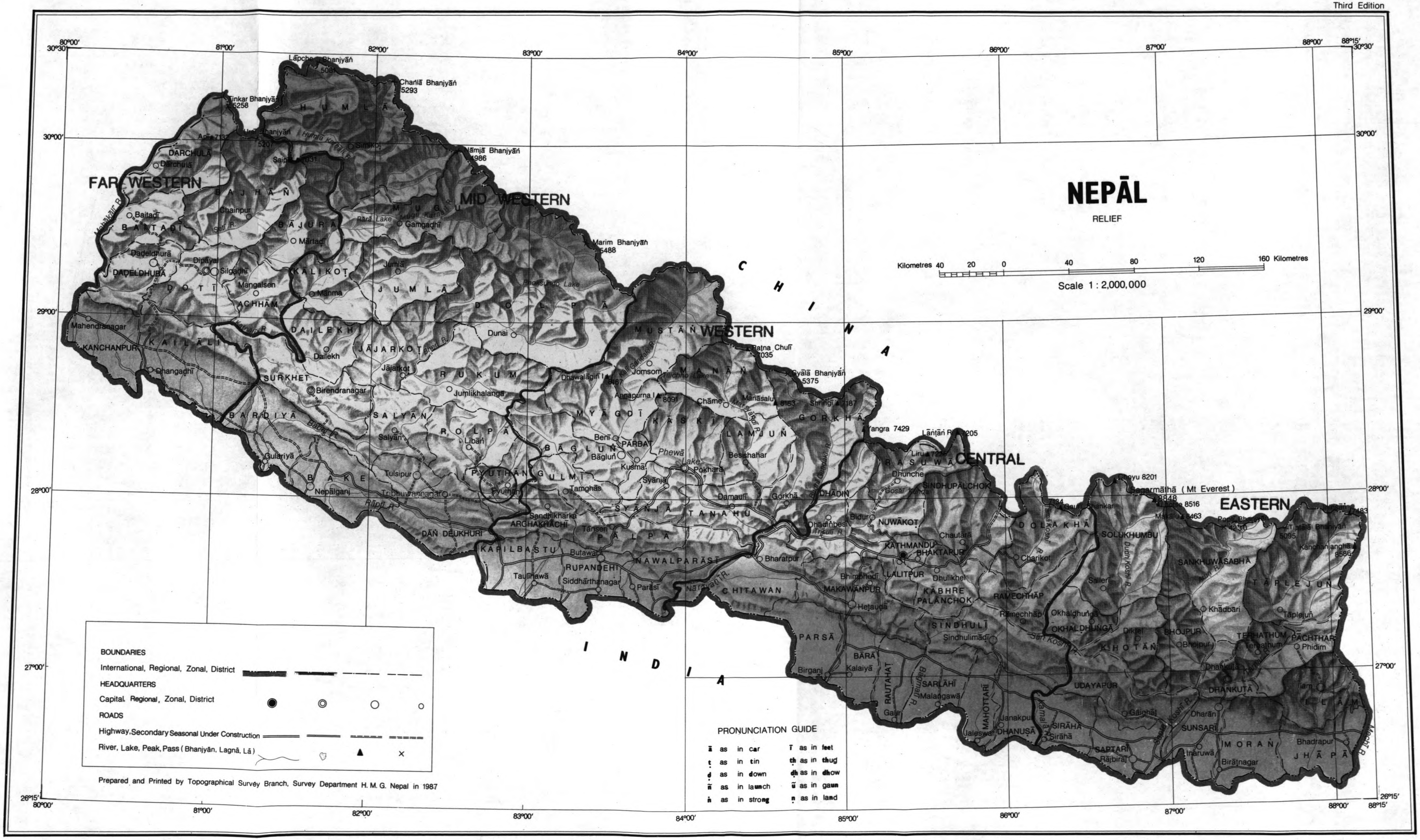

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a	grid bar opening, height of radial gate axis
b	width (perpendicular to flow)
c	discharge coefficient, roughness coefficient (Hazen-Williams)
d	grid bar spacing, grain diameter
e	excentricity
f	correction factor
g	gravity constant
h	water depth, sill height
l	length
m.EL	meters elevation above sealevel
p	pressure, piezometric head, intake sill height
q	specific discharge
r	gate radius

s	slope
v	velocity
w	settling velocity, gate opening, weight/m²
x	horizontal coordinate (in direction of flow)
y	vertical component of water depth
z	vertical coordinate
A	area
B	width (perpendicular to flow)
BWL	backwater level
C	sediment concentration
D	diameter, intake trench depth
E	specific energy
F	Froude number, force
FL	floor level
FWL	flood water level
G	sediment discharge, alternative Froude number
H	energy head
L	length (in direction of flow), lifting force
M	Mannings constant of roughness, bending moment
MW	Megawatt
N	normal force
Q	discharge
R	(hydraulic) radius, storage
Re	Reynolds number
S	shear force
SBL	sluiceway bottom level
T	torsion
TWL	tailwater level
U	velocity in sediment transporting pipeline
US $	United States dollar
W	weight
WL	water level
α	angle
β	crosswave front angle, Tyrolian intake grid slope
γ	safety factor
Δ	weight of sand in water
Δx	change of x
Θ	radial gate lip angle, angle of sluiceway contraction
κ	discharge coefficient (Frank)
μ	contraction coefficient
ν	kinematic viscosity
ξ	coefficient for expansion or entrance losses
ρ	specific weight
σ	normal stress
τ	shear stress
$\emptyset$	coefficient for rack losses, angle of internal friction
ω	reinforcement percentage

Indices:

a	steel
avg	average
b	(desilting) basin
c	critical, canal, concrete
e	entrance, expansion, extra
f	friction, flushing, floor
g	gate, grain, ground
H	horizontal
i	intake
j	jump
m	mean
p	pipe
r	river
s	sand, sluiceway
t	turbine, trench, total
T	(sediment) transport
V	vertical
w	water, wall, weir

1. INTRODUCTION

1.1. CONTEXT AND HISTORY OF THE PROJECT

The potential for hydro power in Nepal is immense; the country clings to the south slopes of the Himalayas and has numerous streams and rivers that are fed by the monsoon rains and the melting mountain snow. The total potential is estimated at 83,000 MW, and even if only a fraction of this is economically feasible to develop it sets an aim for several generations to come. Nepal is one of the poorest countries of the world however, and more than 90 % of its people are subsistance farmers. The rural areas have very limited energy consumption, almost all of it being fire wood, which is ruining the woods and creating an immense erosion problem.

For these reasons His Majesty's Government of Nepal must keep a delicate balance in its development plans, taking into account the scarcety of capital, the lack of know-how and experience, the potential for urban and rural industries, the local needs of the people etc.. Goals have been set for the development of hydro power on all levels of scale, using different types of technologies and various methods of management. Up to now hundreds of micro turbines are installed and under operation by private enterprise. Small power stations under operation by the Nepal Electricity Authority are located at remote places, serving the district headquarters pending the arrival of a connection to the central grid. Bigger schemes (up to 70 MW) are built to serve the national grid and the big cities. Almost all of these are run-of-river systems. Only recently plans for an large reservoir project (maybe 10,000 MW) are reaching a final stage.

In this context the Jhimruk Hydro-electric Project has been studied for some time, to assess its economic feasibility and contribution to the national electricity supply system and the electrification of Pyuthan district. A preliminary study was completed in 1984, concluding that a plan including a rockfill dam of 14 m. height was not feasible. Since then a further study under the responsibility of the NEA has shown that considerable cost reductions can be achieved by using a much lower weir, and daily pondage for the dry months (Jhimruk Hydel Project, Feasibility Study Final Report 1987). Economic feasibility is still marginal though: a benefit/cost ratio of 1.1 (with a discount rate of 12 %).

This report was forwarded to the Butwal Power Company, to see whether they would be interested in developing this project and arranging its financing. BPC is a Nepal based company with expatriate personnel and financial input through its connections with the United Mission to Nepal and aid agencies like NORAD (Norway). BPC has built and operated a similar project in Butwal, and is at the moment constructing one in Syanja District (Andhi Khola Project).

Its consultancy division Hydroconsult was asked to comment on the engineering aspects of the NEA study.

The Hydroconsult team soon came up with more or less serious criticisms on the designs that were proposed. These included comments on the intake design, the tunnel, the desilting basin, the penstock and the powerhouse, as well as the mechanical and electrical equipment. The most fundamental problems involved the intake and the desilting arrangement. It was decided that BPC Hydroconsult would come up with an alternative design.

1.2. SCOPE OF THIS STUDY

The object of this report is:

- to summarise the criticism on the NEA design
- give suggestions for improvements
- present an alternative intake and desilting design
- give a cost estimate of this alternative.

The limitations are that the essential features of the project should remain the same:

- the same location for weir and powerhouse
- the same generating capacity
- the same height of weir
- the same conditions for failure
- no higher costs.

Regarding the last point it can be argued that the proposed design is not a correct standard for the costs, since it will not work properly without considerable and costly alterations. But the economic benefits are so marginal that the project cannot afford any major increases in costs. Since the features mentioned above constitute the most important factors for costs and benefits they are accepted as given limitations. The available resources did not allow the study of a completely different project in that area anyhow.

Necessary data were taken from the NEA report. Where these were insufficient an educated guess had to be made based on experience in Nepal and elsewhere. Two visits were made to the site for some general investigation. More extensive measurements and model tests are in preparation, in cooperation with the University of Trondheim (Norway). **The designs presented in this report can be seen as reasonable approximations that will be tested and optimised through hydraulic model studies in Kathmandu, Nepal.**

This study is part of my fulfilment of the requirements for a Masters Degree in Civil Engineering at the Delft University of Technology (The Netherlands), and as such the responsibility of the author only.

Delft, october 1988 **Peter J. Plooy**

2. PROJECT DATA

[Information in this section has been taken from "Jhimruk Hydel Project, Feasibility Study Final Report" Nepal Electricity Authority , Kathmandu (1987)]

2.1. THE NEA DESIGN

2.1.1. Summary of feasibility study

The Jhimruk Project is essentially a run-of-river project with a daily peaking capability. The development is located on the Jhimruk Khola, one of the two main tributaries of the West Rapti river; the other tributary is the Mari Khola (see figure 2.1). The project diverts the waters of the Jhimruk Khola through a short tunnel and penstock to a powerhouse on the Mari Khola (see figure 2.2). Generating capacity would be 10.5 MW, developed from a net head of 180 m. and a plant flow of 6.9 cumecs. The peaking capability of the project is obtained from an active storage of 82,000 m^3. This storage would be obtained during the dry months through the use of 0.5 m. high removable wooden flashboards. If necessary the plant will be shut down during the main off-peak periods so as to allow the small pond to be filled. The project would supply power to the district headquarters of Pyuthan, Gulmi, Rolpa and Argakhanchi.

Early in the investigations an alternative dam site about 3 km. downstream of the presently selected site was investigated. Although this alternative dam site is only 50 m. wide, the tunnel would be 2000 m. longer and about 20 m. of head would be lost. In this study the scheme using the short tunnel has been adopted.

The mean annual rainfall over the catchment area has been calculated to be 1792 mm. The reference hydrological station is about 10 km. downstream of the dam site. There are records available from 1965 to 1984. The long term mean flow at the dam site is 25.3 cumecs, August having the highest mean flow of 81.6 cumecs, and May the lowest with 3.2 cumecs. The lowest flow recorded was 0.2 cumecs in May 1966.

The project lies in the Pyuthan phyllite zone of the Lesser Himalayas. The area is characterised by dark and light coloured banded grey slate with intercalation of thin layers of calcareous materials. The rock is highly jointed and fractured and is of low strength because of deep weathering. The 300 m. wide river bed at the dam site is covered with alluvium of thickness varying from 7 to 18 m.

Geotechnical investigations were made to check the quantity and quality of construction materials. The results showed that there is an ample supply of suitable materials, impervious and granular, close to the dam. However, a search is needed to locate additional borrow areas for

FIGURE 2.2

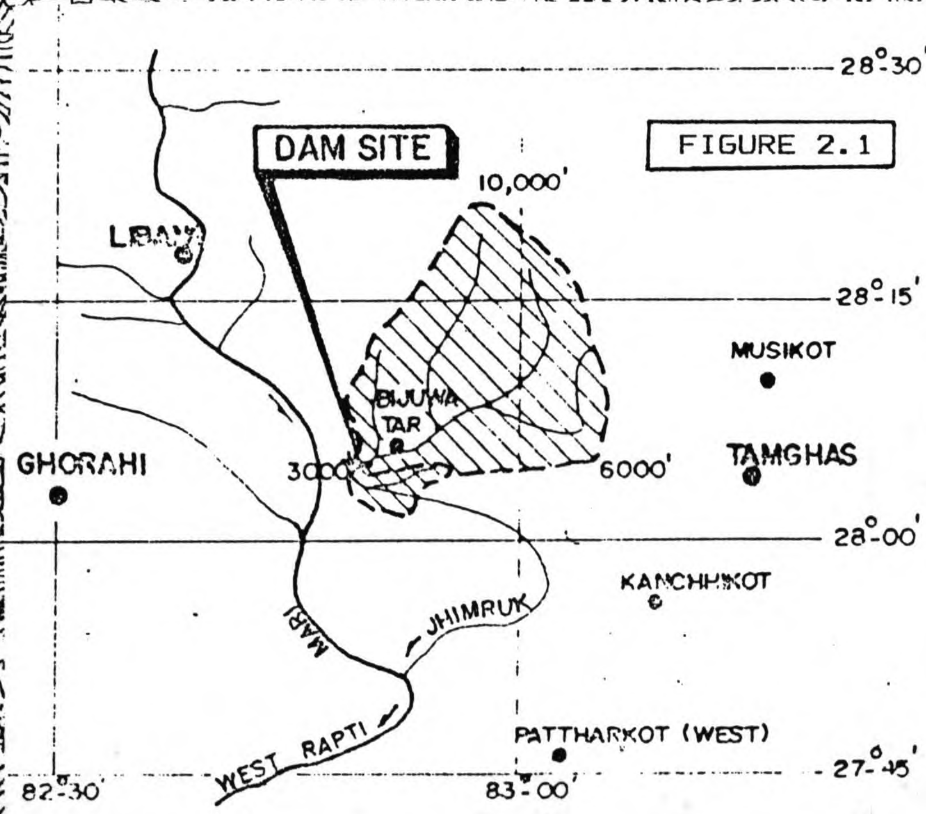

FIGURE 2.1

boulders which would be used to construct the low, 3.4 m. masonry dam. Laboratory testing was done at the site and in Kathmandu.

The layout of the project that developed from the study was:

- A 230 m. long masonry overflow weir with a concrete facing and 0.5 m. high removable flashboards.

- A bottom inlet section, 47 m. long, to provide entry of water to the intake deck section.

- An intake deck and intake gate which allows regulated flows to the tunnel.

- A D-shaped tunnel, 2.7 m. in diameter and 838 m. lomg. The tunnel is lined with concrete and would need steel supports during construction.

- A desilting basin to remove particles down to 0.25 mm. in size.

- A small forebay with a capacity of 1600 m^3.

- A 1.35 m. diameter steel penstock, 364 m. long and supported on concrete cradles.

- A powerhouse with 3 x 3.5 MW generating units. Francis turbines under a net head of 180 m. and coupled to horizontal axis generators have been selected.

The tailwater level of Jhimruk is set at 545 m., i.e. above the top water level of the possible downstream multi-purpose project "Naumuri". If Naumuri were to be built there would be a tailwater reservoir and the expansion of Jhimruk for pumped storage duty should be investigated. This possibility was briefly examened in this study, but it was felt that this aspect of Jhimruk should best be defined when, or if, the Naumuri Project would be built. The feasibility is questionable however, in view of the small storage capacity that is available.

Although no detailed study has been done, preliminary study shows that any negative environmental or socio-economic effects would be minor.

The capital cost of the project, including engineering management (12 %) and contingencies (20 % civil and 10 % electro-mechanical) was estimated to be US $ 15,574,400 (1987).

The economic evaluation of the project, including a sensitivity analysis, showed it to be economically viable. The sensitivity test was done by increasing the capital cost by 10 %.

The evaluation gave the following economic indicators:

	Base case	Capital cost * 1.1
Benefit/Cost Ratio	1.31	1.19
Net Discounted Benefits[1]	$ 4,000,000	$ 3,000,000
Internal Rate of Return	12.9 %	11.8 %

The pondage which has been provided would not add any **energy** (kWh) benefits to the project, but it would provide additional **power generating capacity** (kW) in the dry months of November to June. In May, the driest month, the plant will be able to generate 10.5 MW during 4 peaking hours by shutting down the units for 20 hours; during this month the Q_{90} is 0.92 cumecs. The incremental firm capacity benefit due to pondage is taken to be 8.5 MW.

The annual costs of the wooden flashboards is about US $ 2,550.

Alternative means of supplying power to the district headquarters were examined. These were:
- supply from Jhimruk
- supply grom the grid
- supply from diesel generation.

The latter was found to be the most expensive and supply from Jhimruk the least cost solution.

The project implementation schedule shows a commissioning date for the first unit of September 1993. This allows ample time at the front end for final engineering, decisions to proceed (or not), and for financing. The time allowed from the start of final design to first power available is 53 months.

Conclusions

The Jhimruk Project is economically viable. The district headquarters of Pyuthan, Rolpa, Gulmi and Argakhanchi should be supplied with power from Jhimruk.

The construction of Jhimruk, and a desirable road bridge, would be of considerable benefit to the social and economic development of the area.

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[1] Net Present Value of benefits minus costs over project life cycle.

2.1.2. Salient features of NEA design

HYDROLOGY:

Catchment area	645	km^2
Long term average flow	25.32	cumecs
Diversion flood (construction during dry season)	328	cumecs
Spillway design flood	2500	cumecs

WEIR:

Type	Free overflow, ogee shaped, core of stone masonry with concrete facing	
Maximum height	3.4	m
Flood water level	740.1	m.EL
Crest level	738.0	m.EL
Crest length	230.0	m
Daily pondage	82,000	m^3
Height of flashboards	0.5	m
Nos. of flashboards	65	
Clear opening of flashboard piers	3.05	m

INTAKE:

Type	Bottom intake with sloping glacis	
Length	47	m
Height of flashboards	1.06	m
Nos. of flashboards	14	
Crest level	737.7	m.EL
Trench width	1.6	m
Trench slope	1 : 22.7	

INTAKE DECK:

Length	20.0	m
Width	1.6	m
Slope	1 : 400	

HEADRACE TUNNEL:

Type	D-shaped concrete lined	
Height	2.7	m
Width	2.7	m
Length	838	m

DESILTING BASIN:

Length	30	m
Width	15.5	m
Depth	2.5	m

FOREBAY:

Length	58.5	m
Width	9	m
Depth	1.9	m
Capacity	1600	m^3

PENSTOCK:

Type	Surface	
Length	364	m
Diameter	1.35	m
Material	Steel	

POWERHOUSE:

Type	Surface	
Dimensions	27.6 * 8.5	m
Turbines	Francis (horizontal shaft)	
Net head	180	m
Throat diameter	0.55	m
Speed	1000	rpm
Installed capacity	3 * 3.5	MW
Normal tailwater level	545.0	m.EL
Gross head	189	m

CAPABILITY:

Firm energy	36.8	GWh p.a.
Secondary energy	34.1	GWh p.a.
Total output	70.9	GWh p.a.
Daily peak output	10.5	MW
Peaking capability	4	hours

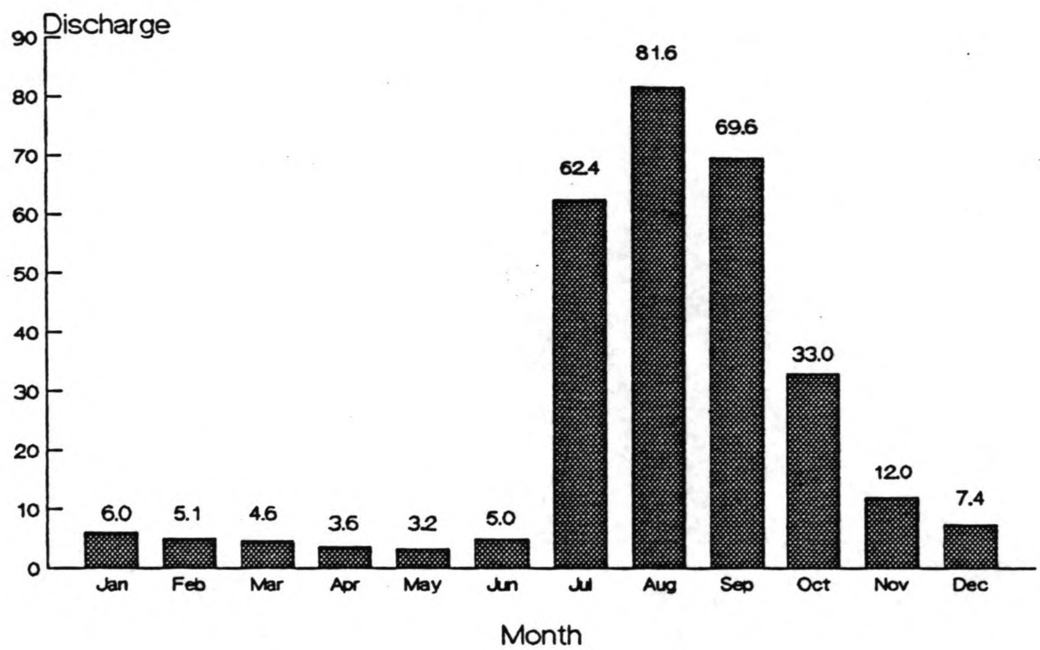
ECONOMICS:

Capacity benefit	55.20	US $/kW
Firm energy benefit	4.98	US c/kWh
Secondary energy benefit	0.54	US c/kWh
B/C ratio at 10 % p.a.	1.31	

2.2. HYDROLOGY

The Jhimruk Khola is a typical rain-fed river, with high discharges during the monsoon and steadily decreasing discharges during the dry season. The four monsoon months (June to September) supply 80 % of the annual inflow. Figure 2.3 shows the mean monthly flows, on the basis of twenty years of gauging.

FIGURE 2.3 Mean monthly discharge [cumecs]



August is the wettest month, May the driest. For these months the flow duration curves are given in figure 2.4.

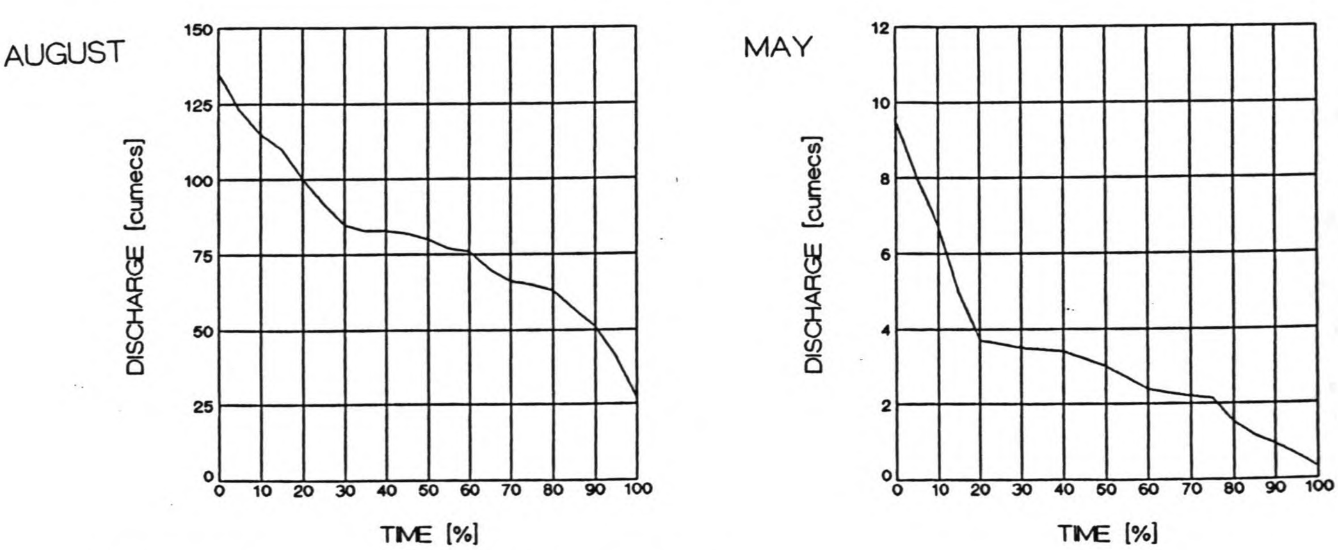


FIGURE 2.4 Flow duration curves

The catchment area consists mainly of steep hill slopes, which results in high flood discharges of short duration. A flood analysis has been done, of which the results are shown in table 2.1.

TABLE 2.1	FLOOD ANALYSIS				
Return Period (years)	Flood Peak (cumecs)	Flood volume (given as average flow) 1 day	2 days	3 days	4 days
5	817	505	434	350	330
10	1021	627	517	418	361
100	1722	979	757	588	613
1000	2441	1340	982	789	813
10000	3154	1687	1048	894	852

The flood with a 5 year return period is presented graphically in figure 2.5., using the data from table 2.1.

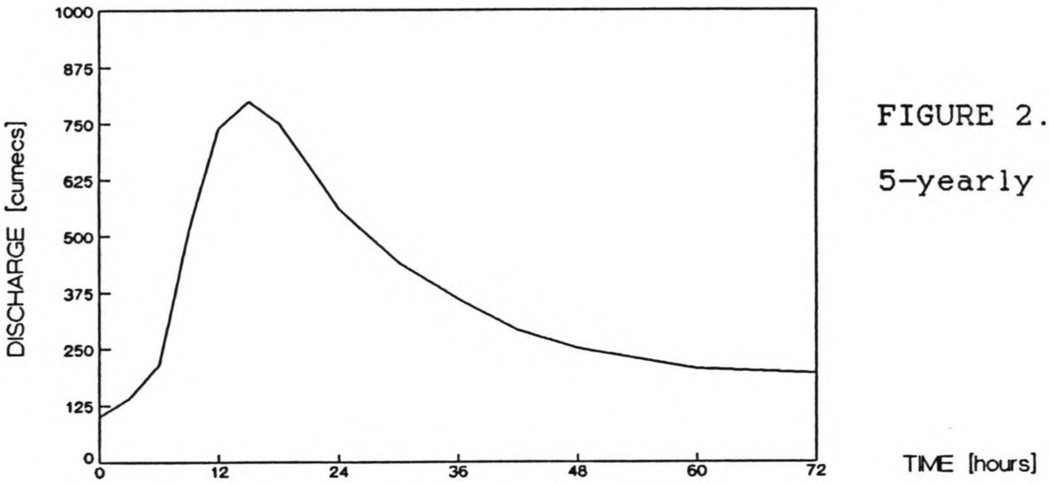


FIGURE 2.5

5-yearly flood

A separate analysis for the floods during the dry season gave the results shown in table 2.2. They are important for the diversion works during the construction.

TABLE 2.2	DRY SEASON FLOOD ANALYSIS
Return period (years)	Flood discharge (cumecs)
5	236
10	328
20	414
50	527
100	612
1000	891
10000	1170

DESIGN FLOODS:

- for operational purposes: 500 cumecs (1:5)
- for structural safety: 2500 cumecs (1:1000)
- for diversion works: 330 cumecs (dry season 1:10)

The relationship between discharge and water depth at the dam site is given in figure 2.6 and table 2.3. The backwater level is computed with the formula for a short-crested weir:

$$Q = \mu * 2/3 * \sqrt{(2g)} * H^{1.5} * B$$

The tailwater level is computed with the Chezy formula:

$$Q = 34.5 * \sqrt{s} * h^{1.5} * B$$

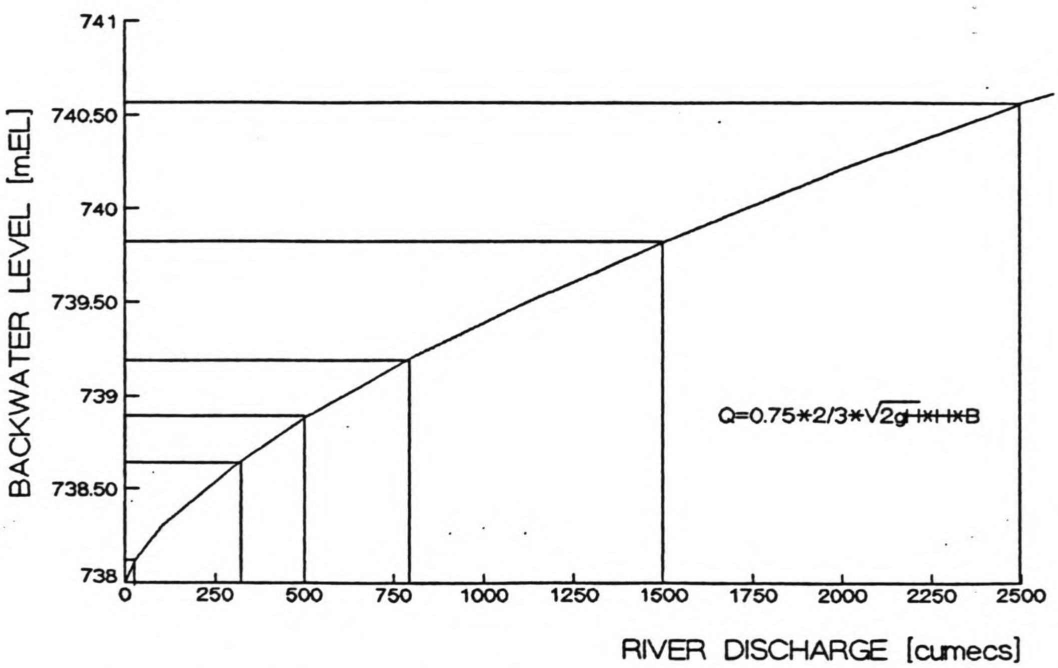


FIGURE 2.6 Waterlevels just upstream of weir

TABLE 2.3 RIVER DISCHARGES AND WATER LEVELS AT WEIR SITE		
Discharge (cumecs)	Backwater level (m.EL)	Tailwater level (m.EL)
30	738.12	736.00
50	738.20	736.07
100	738.30	736.16
300	738.62	736.45
500	738.88	736.67
800	739.20	736.96
1000	739.39	737.13
1500	739.94	737.62
2500	740.56	738.16

2.3. GEOLOGY

The project site lies in the Pyuthan phyllite zone of the Lesser Himalayas. The rock has thin layers and is highly jointed and fractured. It generally trends NW - SE, and dips towards the NE at angles of 40° - 70°. Deep weathering along the joint planes causes low strength. The region is seismically active, but no concentration of earthquake epicentres was found near the project site.

The topography is quite rugged. Sheet erosion, gullies, rock falls and landslides are frequently observed. More than 70 % of the area is covered by thick talus materials. At the dam site the right bank slope is steeper than the left bank slope (40° - 50°). Rock falls and landslides are present 100 m. downstream on the right bank. However, rock exposures are present in both abutments at the dam axis. The **dip of the rock** at the right bank is advantageous (inward).

The river valley along the dam axis is covered by river alluvium of 7 - 18 m. thickness. Its **permeability** has been measured at field level (1 m. deep), and has an average value of **k = 0.1 cm/s**. The **bearing capacity** at 1 m. depth is: **50 kN/m²** at the right bank,and
500 kN/m² 15 m. from the right bank.

The riverbed profile at the dam site and 100 m. up- and downstream has been reduced from 1 : 500 survey maps, and are shown in figure 2.7. The **average level** at the dam axis is **735.9 m.EL**, and the **average slope** is **0.6 %**.

The penstock alignment is located along thick talus deposits, sparely wooded. The underlying rock has an outward dip. Slope stabilisation will be necessary. A rock outcrop above the Mari Khola creates a suitable site for the powerhouse.

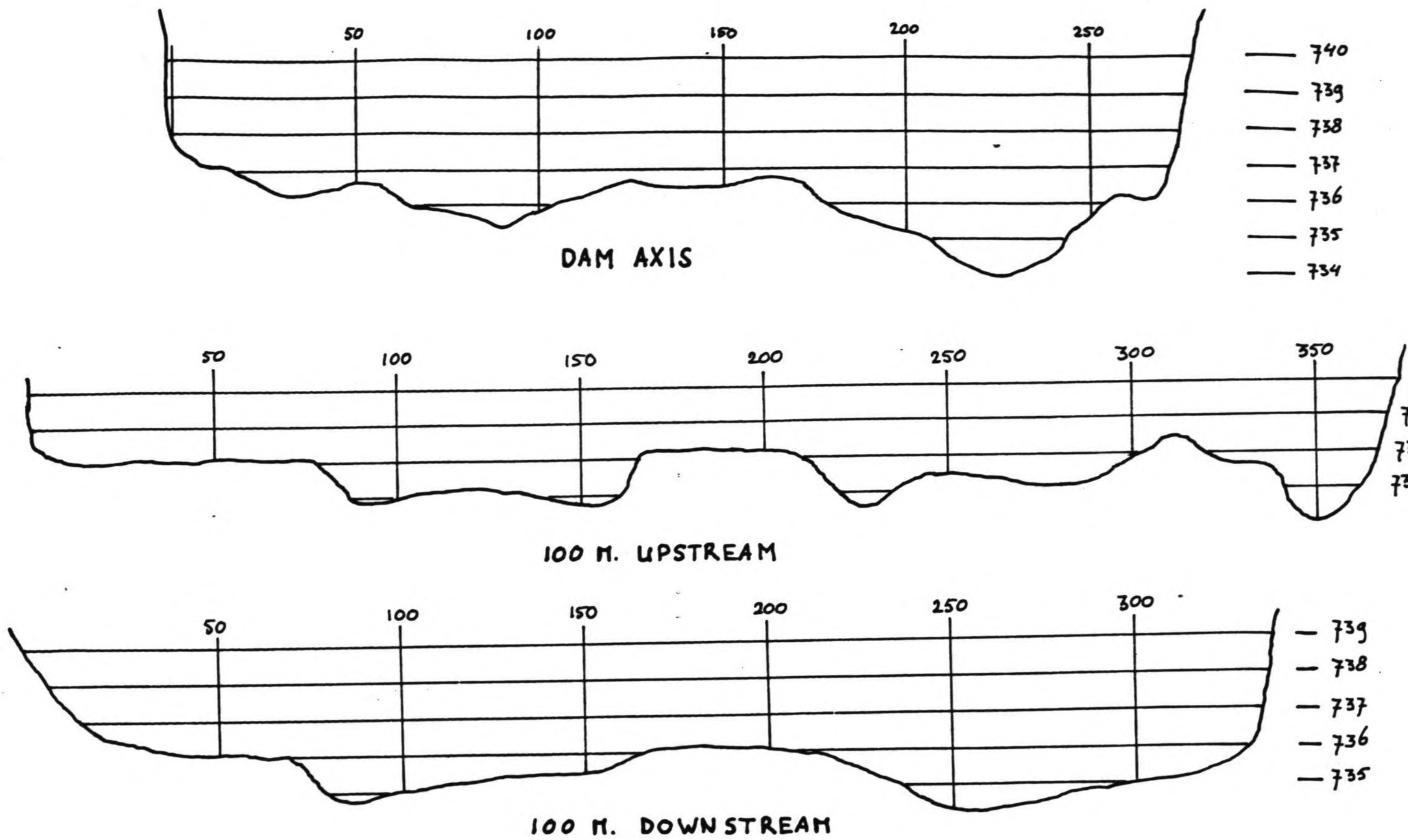


FIGURE 2.7 River bed profiles

2.4. POWER DEMAND

A load survey has been done to get estimates of the development of the power demand. Load centres are the district headquarters and a number of villages with sufficient population density or development potential. The load forecasts have been calculated for five sectors:

- domestic load
- industrial load
- commercial load
- administrative load.

Potential consumers have been counted, and their power demand estimated. The results are shown in table 2.4 and figure 2.8.

TABLE 2.4 LOAD DEMAND FORECAST (kW)					
Year	Domestic	Comm.	Industr.	Admin.	Peak
1993	1039	253	121	305	1322
1998	1327	495	242	313	1828
2003	2135	733	362	321	2732
2008	2726	972	482	330	3470

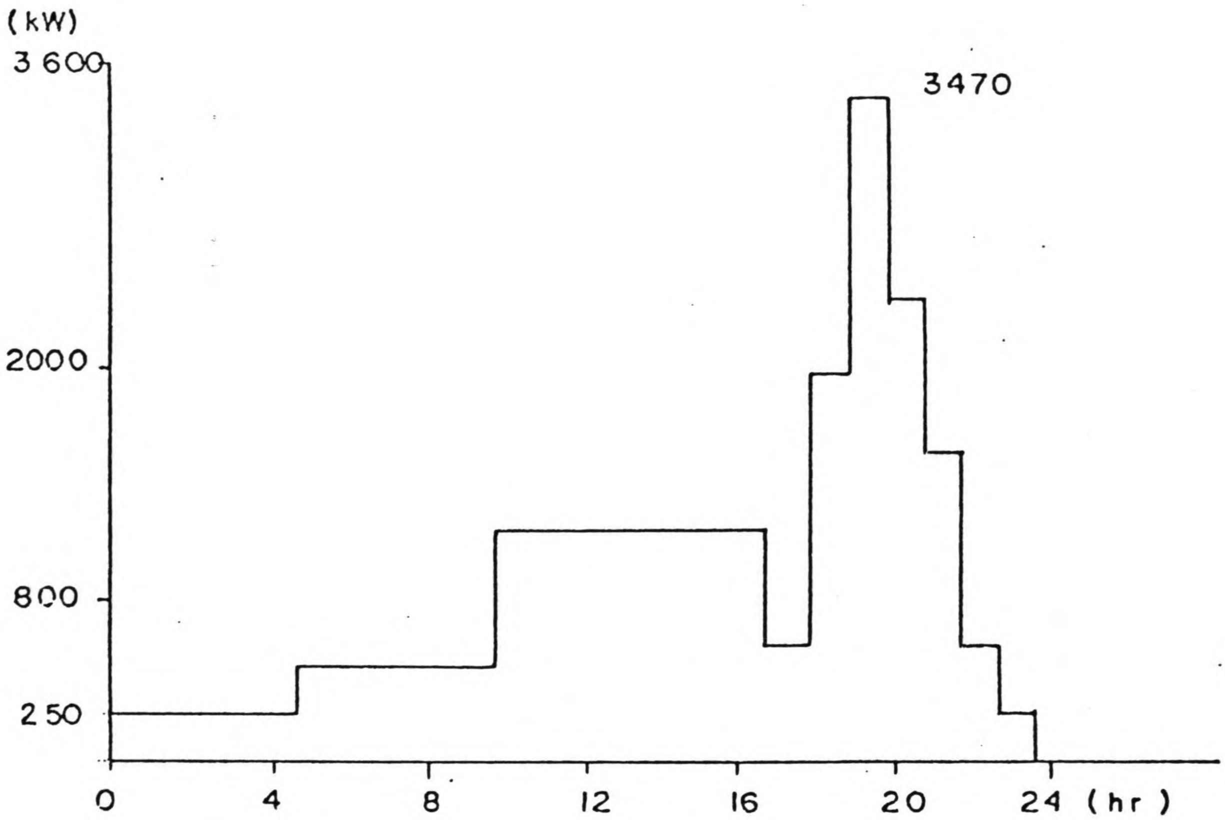


FIGURE 2.8 Daily load cycle

The Jhimruk Hydel Project will be a run-of-river project with only daily pondage. The benefits of Jhimruk for the National Grid has been evaluated, using the power simulation program POWSIM. A distinction has been made between:

- firm capacity benefits 55.20 US $/kW/year
- firm energy benefits 4.98 US c/kWh
- secondary energy benefits 0.54 US c/kWh.

On the basis of these figures a Benefit/Cost ratio was calculated for four different plant sizes, and the optimum capacity was established to be 10.5 MW.

The resulting **B/C ratio** is: **1.31** for discount rate = 10 %,
1.09 for discount rate = 12 %.

3. SOME COMMENTS AND SUGGESTIONS

3.1. INTRODUCTION

A review of the Jhimruk Hydro-electric Project Final Feasibility Report (NEA, 1987) by BPC Hydroconsult showed several weak points in the design of the project. Comments from Norwegian and Dutch consultants confirmed this. In this chapter the comments are summarised as far as the civil engineering aspect is concerned, and suggestions for alterations of design are given.

3.2. GENERAL LAY-OUT

The main problem of the NEA design is the location of the desilting basin: in the hillside, between tunnel and penstock. This location entails:

- high velocities (2.7 m/s) in the tunnel, necessary to avoid depositing of sand
- loss of head because of these high velocities
- strong erosive effect of silty water at high speed; will necessitate expensive concrete lining of the tunnel
- an automatic gate at the intake to avoid overflowing of the desilting basin during turbine shut-down
- deep cutting in the potentially unstable slope of the hill at the basin site
- increased landslide hazard due to the flushing of silted water down the hillside.

These problems can only be remedied by relocating the desilting basin to the right bank of the Jhimruk Khola, so the water is desilted **before** entering the tunnel. The tunnel can then be set at a smaller slope, and be provided with a surge shaft in stead of a spillway, so there will be no danger to the hill slopes and no need for an automatic gate.

The intake type and arrangement will also have to be altered considerably. This problem is the main subject of this study, and will be addressed in the following chapters.

3.3. WEIR

The NEA design provides for an overflow weir, with its crest 2 m. above the natural river bed level (see Drawing 2). It is 230 m. long and made of grouted boulders covered with 300 mm. thick concrete. It is fitted with piers for the 0.5 m. high flashboards that will provide the daily storage of 82,000 m^3. The weir is ogee shaped, and has an 8 m. long stilling basin fitted with chute and floor blocks.
Behind the stilling basin a launching apron is envisaged, consisting of graded filters, concrete blocks and gabions.
Cut off walls are to be constructed both at the upstream and at the downstream end.

3.3.1. Storage

It is questionable whether the 0.5 m. height of the flashboards will provide the required active storage. Based on the available maps it is estimated that the storage capacity would be no more than 50,000 m^3. It is advised to check the calculations concerned on the basis of more detailed survey maps.

Some 20 ha. of cultivated land will be lost in the pondage area. It is suggested that about half of this area, which has a higher level than the other half, can be reclaimed after completion of the project. This can be achieved by building dykes around it, flooding this area periodically with silted water and allowing the silt to deposit. The cultivated land can thus rise in level simultenously with the river bed, until the weir crest level is reached.

The reservoir area will thus be smaller, and the **flashboards must therefore be higher**. The extra costs will probably be offset by the reduction of compensation payments to the farmers. Anyway, the reduction of loss of cultivated land is worth the effort.

3.3.2. Weir construction

It is doubtful whether constructing the weir with grouted boulders is a favourable method. Quality control will be a problem, and it will require more (expensive) cement than masonry would. It is therefore advised to use **stone rubble masonry** for the weir.

It should be considered whether the concrete lining can be done without. This would depend on the attacks on the weir crest by rolling boulders etc. Because of the river width and gentle slope this will not be as big a problem as elsewhere in Nepal.

The "nose" at the weir crest is an unnecessary complication, and should be deleted.

3.3.3. Stilling basin

It is suggested that the floor blocks in the stilling basin could wear down too quickly in the fast flowing silted water or damaged by rolling boulders. If they are not repaired in time this would result in the hydraulic jump traveling downstream and doing considerable damage there.

This problem can be solved by deleting the blocks and increasing the size of the stilling basin. It should be looked into in more detail whether the extra safety is worth the extra costs, on the basis of experience with similar weirs elsewhere in Nepal.

3.3.4. Bed protection

The rather elaborate design of the bed protection behind the weir can be simplified. A graded filter protected with gabions should be enough, if underseepage is small enough. This can be achieved more cost effectively with a clay blanket upstream of the weir than with cut off walls.

.3.4. TUNNEL

The tunnel can have a smaller section and slope than the NEA design has: 2.7 * 2.7 m. and 1 : 270. When the slope is set at 1 : 1000 the section can be 2.5 * 2.5 m., which will result in a velocity of 1.4 m/s at full flow.

With clean water and relatively low velocities the lining if the tunnel can be masonry instead of concrete. This will result in considerable cost savings, and easier construction methods known by Nepali contractors.

Together with the changes in the intake and desilting arrangement (see next chapters) this tunnel design results in a reduction of the cumulate head losses by 6.5 m. (= 3.6 % of the total head).

3.5. PENSTOCK

The penstock is an on surface pipeline, supported by concrete saddles and anchor blocks. To avoid many sharp bends this involves cuttings into the hillside. This could result in rain water taking the same course and causing undercutting and landslides. A more detailed survey should be done to assess these hazards and find ways of protecting the slope.

It is suggested that the penstock be taken underground, to avoid problems with the slope stability. This is a very expensive solution however, and based on the survey map and a quick field survey the cuttings in the current design do not seem to be large enough to justify such a drastic step.

3.6. POWERHOUSE

The powerhouse is located on a little plateau 15 m. above the Mari Khola level. This is done because of the possible future implementation of the proposed Naumuri Multi-purpose Project. This project would create a reservoir in the Mari Khola valley, with a maximum level just below the proposed level of the Jhimruk powerhouse.

This is an unnecessary precaution however, as long as the turbines can be designed to work against back pressures up to 12 m. It is therefore proposed to use Francis turbines, that have this capability. As long as the Naumuri Project has not been implemented the Jhimruk Project can thus use about 15 m. extra head.

The powerhouse will have a superstructure at the proposed site, but with large vertical shafts for the turbines, which are placed 15 m. lower. The turbines will be of the vertical type, because of the advantages this gives when designing the powerhouse to be capable of taking back pressure. Short tailrace tunnels are excavated at level 529 m.EL, which is the Mari Khola dry season level.

These alterations will raise the costs by 3 %, but will add 9 % to the power generating capacity.

4. JHIMRUK KHOLA INTAKE ANALYSIS

4.1. RIVER DIVERSION STRUCTURES

4.1.1. Introduction

Whenever water is to be withdrawn from sediment carrying streams special attention has to be given to the problem of separating the water from the sediment. Especially if the water is used for power generation, since turbines are very sensitive to sand. This separation is usually achieved by a combination of **sediment exclusion** (deflection of sediment away from the intake, prevention of sediment intrusion into the intake) and **sediment ejection** (removal of sediment which has entered the canal with the diverted water). The more efficient the sediment exclusion is, the less costly are the necessary installations for the sediment ejection (i.e. the desilting basin and its flushing arrangement).

The choice of a certain type of intake structure is therefore an important step in the design process, and must be analysed in conjunction with the desilting arrangement, and with the varying river conditions and canal operations.

There are three basic types of intake structures (see figure 4.1):

- lateral intakes
- frontal intakes
- bottom intakes

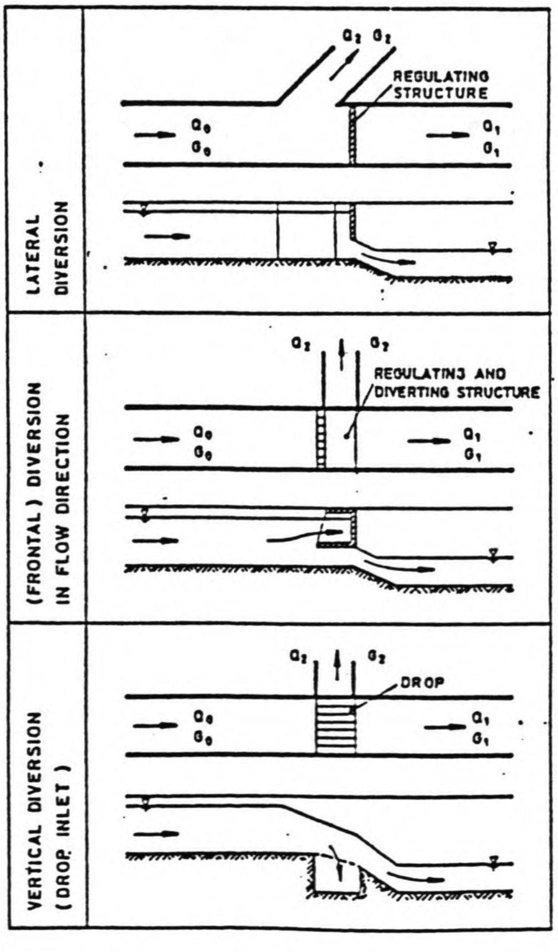


FIGURE 4.1.

Basic types of intake structure

(source: litt.7)

They have different efficiencies of sediment exclusion, especially with regard to the **bed load**. This is the part of the total sediment discharge that moves by saltation, rolling or sliding near to or on the stream bed. The bed load consists of the heavier particles, that are the most difficult to remove after they have entered the canal. An intake design that avoids diverting water from the lower layers of the river flow therefore has considerable advantages.

4.1.2. Lateral intakes

The most common type of water withdrawal from rivers is the lateral diversion from one bank. The canal headworks are usually located adjacent to the regulating structure (weir or dam, with or without sluices). A sluiceway is provided near the canal entrance to facilitate the flushing of deposits in front of the intake.

The performance of these intakes with regard to sediment exclusion is highly dependant on their positioning and the presence of river training works. To avoid an over-proportional amount of bed load intrusion a flow pattern near the bed must be generated having velocity components **away** from the intake. Such conditions exist a priori in a curved reach of the river, where the spiral current creates a favourable location for an intake in the outer bank of the bend (see figure 4.2).

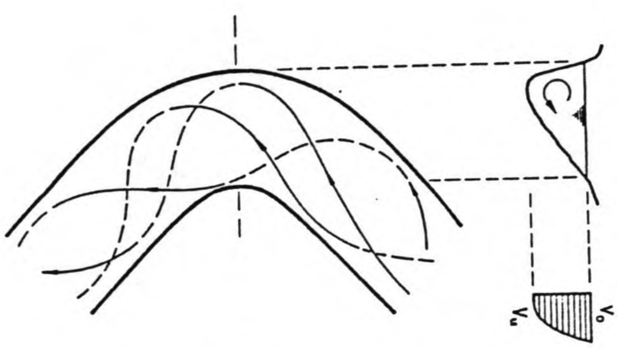
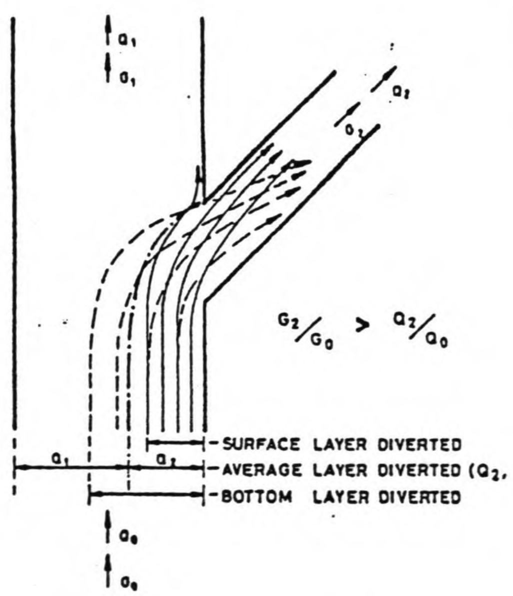


FIGURE 4.2

Spiral flow in a river bend



(source: litt.7) FIGURE 4.3

Flow pattern at a branching point

In straight river reaches the branching canal creates an unfavourable secondary flow, if no appropriate river training works are included in the design (see figure 4.3). Such measures are aimed at enforcing an artificial curvature of flow away from the intake. This can be achieved with (a combination of) spur dykes, guide walls and guide vanes (see figure 4.4).

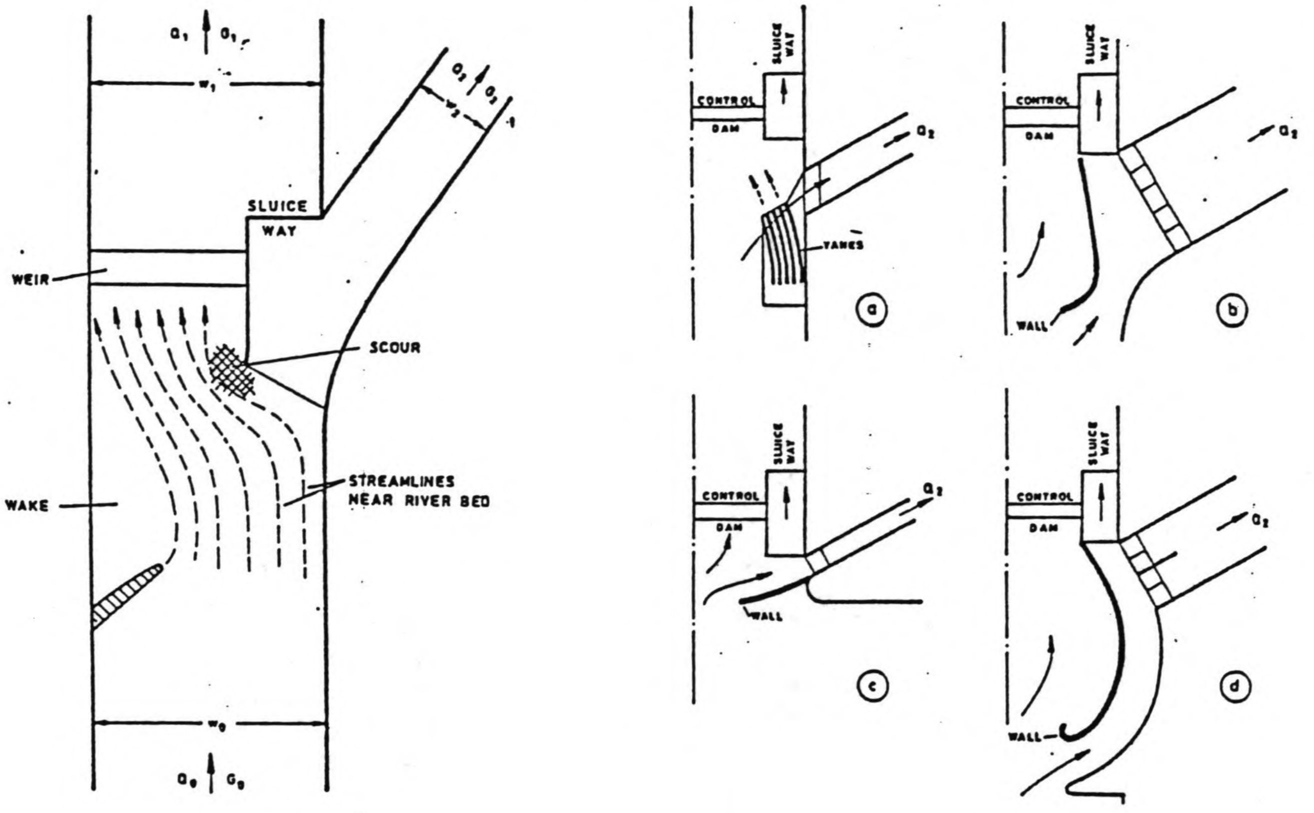


FIGURE 4.4 (source: litt.7)

River training works: spur dykes, guiding vanes and guiding walls.

The best choice of training works can only be found with the help of model studies for individual local conditions. In general it can be stated that guide vanes and walls are more appropriate for wide and shallow rivers.
A different approach is the use of sediment removing devices which draw only the bottom layers of the flow, e.g. tunnels underneath the canal entrance. The head drop over the control dam should be enough to facilitate this flushing (see figure 4.5).

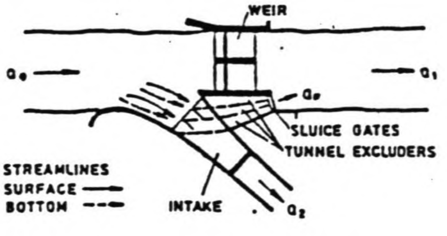


FIGURE 4.5

Lateral intake with tunnel excluders (source: litt.7)

4.1.3. Frontal intakes

There are two basic types of frontal or direct intakes:

- pier-type direct intake
- direct intake with undersluice

The first type is based on the flow pattern around bridge piers, which creates a scour hole in front of such an obstruction. An intake incorporated in the nose of such a pier facing directly into the oncoming flow will reject practically all bed load. Figure 4.6 shows a typical arrangement.

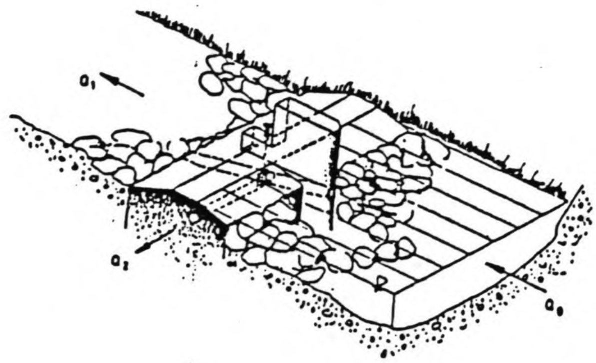


FIGURE 4.6

Pier-type direct intake

(source: litt.7)

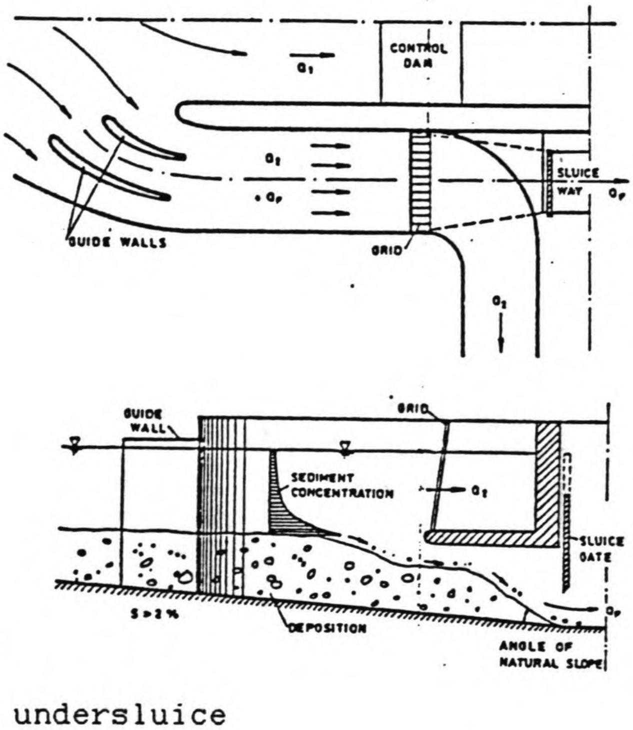


FIGURE 4.7

Direct intake with undersluice

Because of the relatively small dimensions of the entry in the pier this type is not suitable for wide and shallow rivers.

The arrangement of a frontal intake with **undersluice** is shown in figure 4.7. The basic design principles are:

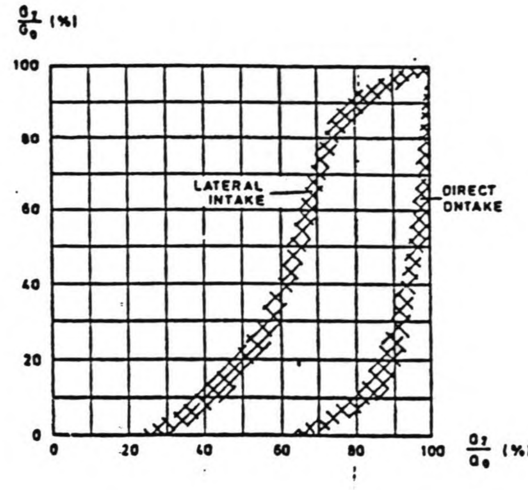
- establishment of parallel and uniform flow in the sluiceway, with guide walls at the entrance and sufficient length to reduce turbulence and promote settling of the heavier particles,
- continuous flushing through the partially opened undersluice,
- diversion of water from the upper layers,
- distance between grid section and sluice section as small as possible,
- complete flushing of sluiceway during flood flows.

A comparison of sediment exclusion for direct intakes and lateral intakes shows that higher proportions of the river flow can be diverted by direct intakes without large amounts of sand entering through the grid (see figure 4.8).

FIGURE 4.8

Comparison of sediment exclusion

(source: litt.7)



A disadvantage of frontal (or direct) intakes is the fact that its feasible dimensions limit the absolute amount of diversion discharge. Especially in rivers with high flood levels it is not wise to expose large structures frontally to the force of the river flow. Another problem is the blocking of floating materials by the grid, which makes frequent cleaning necessary.

4.1.4. Bottom intakes

This type is also known as drop inlets or Tyrolian weir. It is often used in steep mountain streams, diverting discharges up to 15 cumecs. It consists of (figure 4.9):

- a trench, built into a weir, covered by a grid which slopes in the direction of the stream flow
- a flushing pipe returning to the stream bed, with a gate or valve
- a diversion canal.

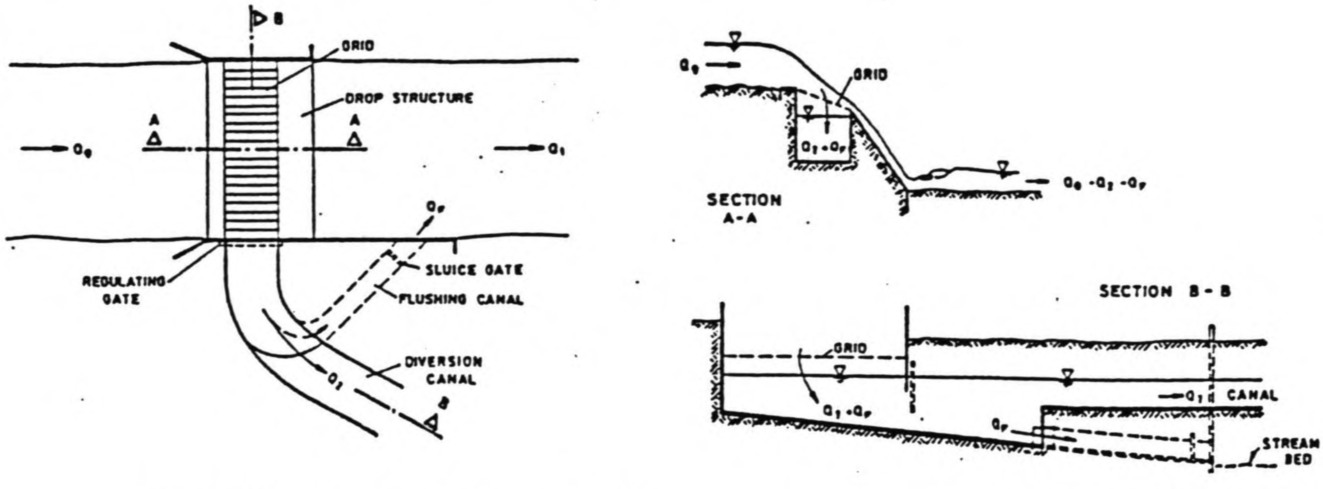


FIGURE 4.9 Tyrolian weir (source: litt.7)

The diverted discharge and all bed load particles smaller than the grid openings enter the trench. The excess discharge and the bigger cobbles and stones pass over the grid. The flushing discharge draws the sediment-laden bottom layer of the diverted water through the flushing pipe.

Important **design considerations** are:

- the width of the intake must be enough to allow sufficient diversion under low flow conditions
- the spacing and cross section of the grid bars should be such that both entering of large particles and choking of the grid are avoided
- the grid bars should be strong enough to carry the weight of rolling stones
- velocities in the trench should be high enough over its whole length to avoid its filling up with sediment
- the head drop over the flushing pipe should be enough to facilitate sufficient flushing under flood conditions.

The fourth point implies that free surface flow must be maintained in the trench. Under pressure flow (no free surface) only a short length of the trench has sufficient flow velocity to allow sediment removal (see figure 4.10). For short trenches this is not a problem, but longer ones will clog during periods of pressure flow.

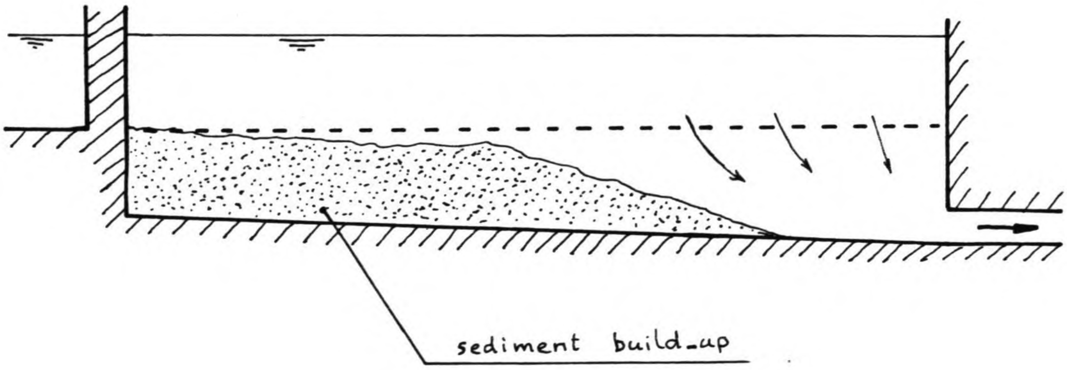


FIGURE 4.10

Drop inlet - under pressure flow conditions

These design considerations lead to the conclusion that bottom intakes are not suitable for wide and shallow rivers, rivers with big differences between low flow and high flood conditions, and where insufficient head drop is available.

4.2. NEA DESIGNED TYROLIAN INTAKE

4.2.1. Main problems

The intake as designed by the NEA is of the Tyrolian type. The inlet section of the weir is 47 m. wide, the remaining 230 m. of the weir being the overflow section. An intake gate regulates the flow into the tunnel, its operation being connected to the turbine demand. Figure 4.11 shows sections of the intake trench.

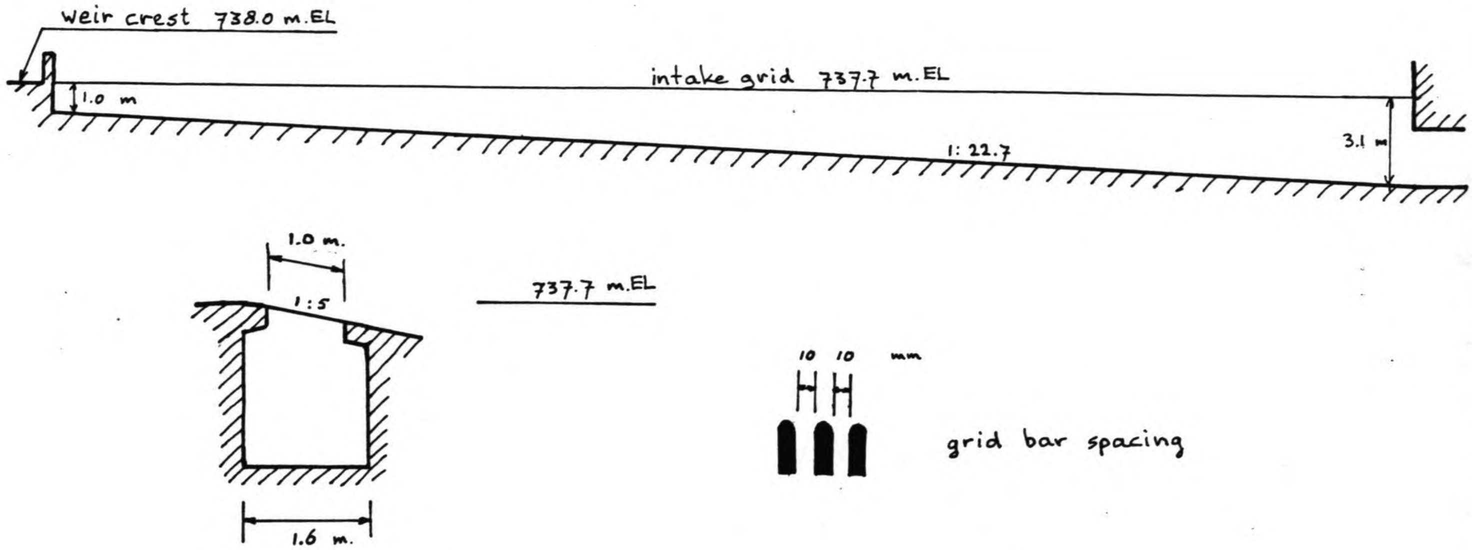


FIGURE 4.11

NEA designed Tyrolian intake

The design does not incorporate any sediment flushing features. All sediments entering through the grid will be transported into and through the tunnel, to a desilting basin on the Madi Khola side. The intake gate will have to match the flow into the tunnel with the turbine discharge plus the flushing discharge of the desilting basin.

During the dry months free surface flow is possible in the collecting trench, but during the monsoon pressure flow is expected. The NEA designers seem to overlook that with their design free surface flow can never be maintained when the turbine discharge is low, since the water entering the trench cannot be diverted anywhere in that case.

From this short description the following **problems** can be identified:

1. The absence of a flushing arrangement and the impossibility of maintaining a free surface in the collecting trench will quickly lead to the silting up of the trench, since only a small part of its 47 m. length can be kept open under pressure flow conditions.

2. Any failure of the intake gate to close in response to the turbine demand will lead to excessive spilling out of the desilting basin on the Madi Khola side of the ridge.

The first problem would be acceptable if it would only mean that the sand had to be dug out of the trench once a year after the monsoon period. The plant operation and the river conditions are constantly changing however, and often very fast. Two days after a flood has filled up 80 % of the length of the trench the water level in the river can be so low that 50 % of the intake width is needed to catch the demand discharge [1]. This would mean that after every (serious) flood the intake trench would have to be cleaned in order to achieve sufficient intake capacity again.

The second problem poses a serious risk. Infallible performance of an automatic gate can not be counted upon under the local conditions. This problem can only be solved by locating the desilting basin next to the Jhimruk Khola. The tunnel and penstock can then be built as a closed system, without any danger of spilling. This solution is in accordance with the changes in design suggested in chapter 3.

4.2.2. Possible improvements

It might be possible to improve the NEA design of the intake by adding a flushing arrangement and changing some dimensions. In the following quantitative analysis some assumptions have been taken to one extreme, for the sake of argument.

The magnitude of the silt problem is determined mainly by the following parameters:

```
- width of intake            )
- length of grid             )->   capacity of intake
- bar spacing of grid        )

- width of trench            )->   free surface flow
- depth of trench            )

- diameter of flushing pipe  )->   capacity of pipe
- available head drop        )
```

---

[1] Length and width are defined in relation to the direction of the relevant flow (parallel and perpendicular respectively). In this case the trench length is equal to the intake width.

The design discharges and levels are:

river: Q_{min} = 0.9 cumecs (90 % dependable flow in May)
H_{min} = 738.0 m.EL. (weir crest level)
Q_{max} = 500 cumecs (1 day average of 5 yearly flood)
H_{max} = 739.0 m.EL. (flow over weir)

plant: $Q_{t,min}$ = 0 cumecs
$Q_{t,max}$ = 6.9 cumecs
$Q_{f,max}$ = 1.1 cumecs (15 % for flushing during monsoon)

4.2.2.1. Grid dimensions

Limiting conditions for the capacity of the intake grid are: H_{min} and $Q_{t,max}$. In words: when the reservoir is almost empty the turbines should be able to run at full capacity.

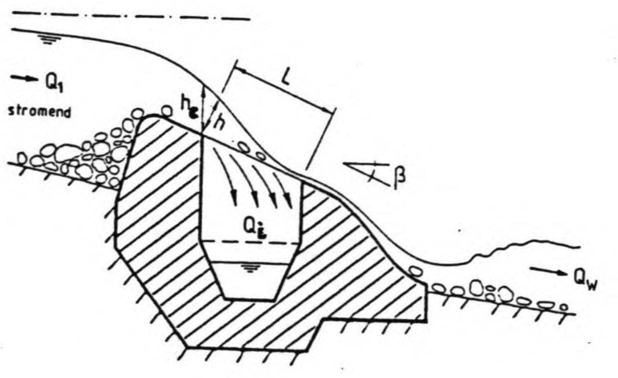
A simple empirical thumb rule states that 2 m^2 grid area is needed for every cumec of diverted water, assuming a bar width : spacing ratio of 1:2, a grid slope between 15 and 30 deg., and provided there is no choking. Usually 5 to 7 m^2 grid area per cumec is taken, to have an ample safety margin [litt.7]. This would lead to a grid of 1 * 14 m^2 as a minimum and 1 * 42 m^2 for safety reasons.

A theoretically more sound basis for design is the formula developed by **Frank [litt.6]**. For complete withdrawal of the discharge flowing over the grid this formula is:

$$Q_1 = 2/3\ c\ \mu\ b\ L\ \sqrt{(2\ g\ h)}$$ (see figure 4.12)

with
$c = 0.6\ (a/d)\ (\cos\beta)^{1.5}$
a = opening width
d = bar spacing
β = grid slope
μ = contraction coefficient
b = grid width
L = min. grid length for complete withdrawal
$h = K\ h_c$
K = coefficient depending on β (table 4.1)
h_c = critical depth

It is assumed in this formula that the initial depth is critical. This is certainly the case under low flow conditions.



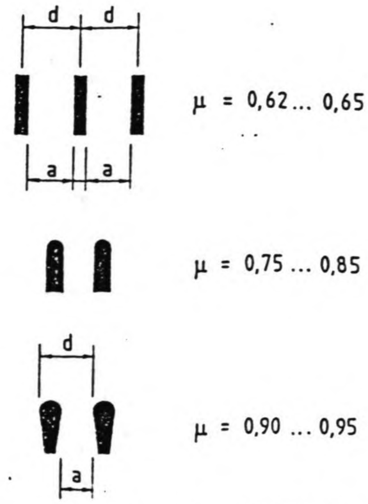


FIGURE 4.12 (source: litt.11)

Variables in Frank's formula

β	0	4	8	12	16	20	24	28
κ	1.000	0.961	0.927	0.894	0.865	0.837	0.812	0.788

TABLE 4.1 Frank's coefficient (source: litt.11)

The necessary width b of the intake can therefore be calculated using the formula for a sharp crested weir [litt.4]:

$$q = 2/3\ \mu\ \sqrt{(2\ g)}\ H^{1.5}$$

and $b = Q_{max} / q$

The minimum energy head H is the difference between the minimum water level (= weir crest level) and the grid level: 738.0 - 737.7 = 0.30 m.

With μ = 0.60 , H = 0.30 m. and Q_{max} = 6.9 cumecs the above formula gives:

b = 24 m.

Entering this value in Frank's formula, and with:

a =	10	mm	c =	0.291	
d =	20	mm	H =	0.30	m
β =	11.3	deg	κ =	0.90	
μ =	0.75		Q_1 =	6.9	cumecs

this gives: L = 1.03 m.

Frank advises a safety margin of at least 20 %. Considering the improbability of regular cleaning of the grid 50 % seems to be more appropriate, which gives:

l = 1.50 m.

4.2.2.2. Dimensions of collecting trench

To keep the trench open it is necessary to maintain free surface flow under all operational conditions. The design river flood for operation is chosen at Q_{max} = 500 cumecs. This means that shutting the plant down for a few hours is accepted once every five years.

For Q = 500 cumecs the water level at the weir is 738.88 m.EL (see table 2.3).

The discharge flowing over the intake section is (approximately):

$$Q_1 = 2/3\ \mu\ b\ \sqrt{(2\ g)}\ H^{1.5}$$

With μ = 0.60
b = 24 m.
H = 738.88 - 737.70
= 1.18 m.

this gives: **Q_1 = 54 cumecs.**

Using this figure in Frank's formula gives as minimum grid length (in direction of river flow) for complete withdrawal: L = 4.0 m.
Since the grid length l = 1.5 m. this means that there is only partial withdrawal. For such cases Frank developed a formula:

$$Q_1 = q_1\ b\ (1 - q_2/q_1)$$

and table 4.2, which gives the ratio q_2/q_1 for a given x = (L-l)/L. In this case:

x = 0.625
q_2/q_1 = 0.365

so **Q_1 = 34 cumecs.**

x	1.0	0.95	0.9	0.8	0.7	0.6	0.5	0.4
q_2/q_1	1.0	0.885	0.784	0.608	0.459	0.333	0.229	0.145

TABLE 4.2 Partial withdrawal (source: litt.6)

The dimensions of the collecting trench should be such that this discharge can be diverted at all times.
The vertical limits are:

- the level of the intake grid (NEA design: 737.7 m.)
- the level of the stilling basin behind the weir.

The stilling basin level is currently set at 735.0 m.EL, which is ± 1 m. below the natural river bed. Setting the flushing pipe outlet more than 0.5 m. lower could cause clogging problems of these outlets. Their level is therefore chosen to be **734.5 m.EL**.
The width of the trench is limited by the weir dimensions, and is taken to be **B = 2 m.** as a maximum.

Figure 4.13 shows these dimensions.

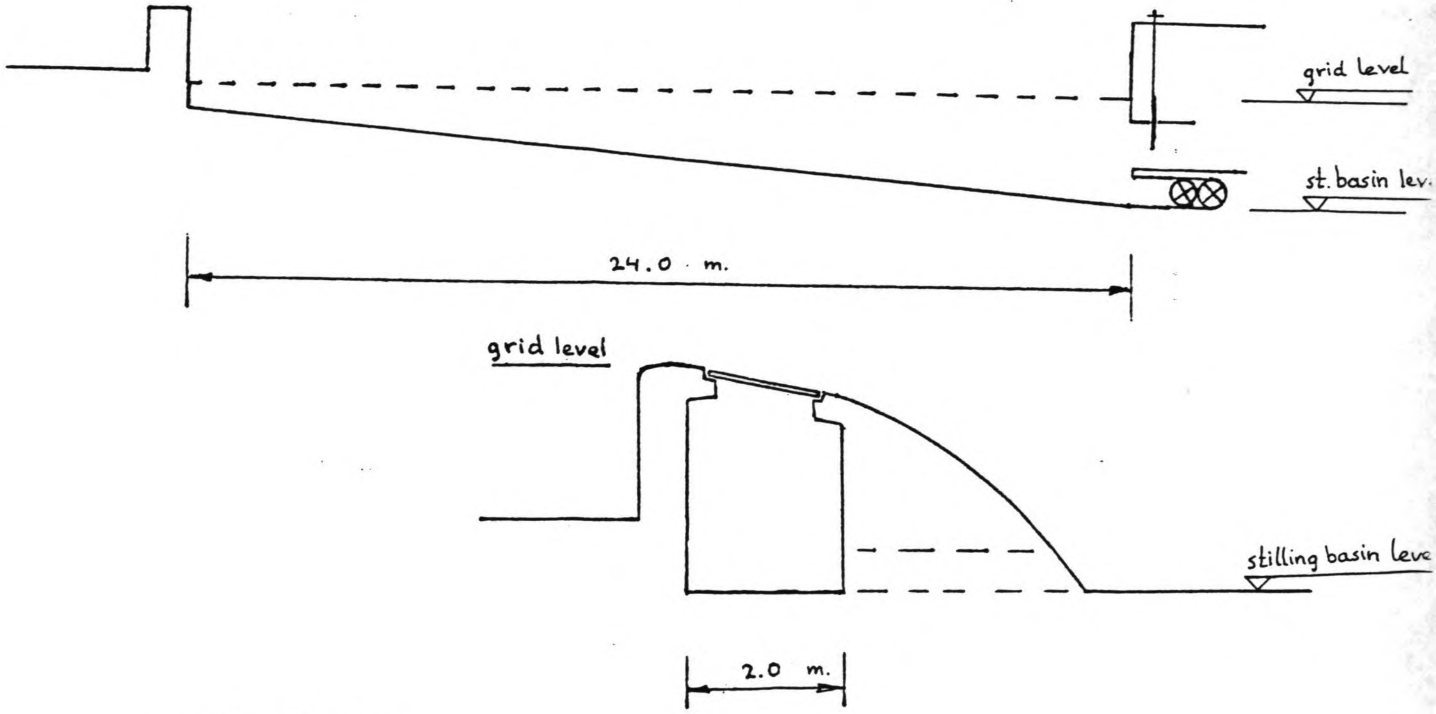


FIGURE 4.13

Trench main dimensions

The hydraulical limits under the design flood conditions are (figure 4.14):

- the tailwater level (C),
- the minimum trench outlet level (B) to enable the maximum intake discharge Q_i = 34 cumecs to be flushed through the flushing pipes,
- the maximum water surface level (A) in the trench.

The grid level must be chosen just above point A, to ensure free surface flow. Point D, the lowest point of the trench, must be just above level 734.5 m.

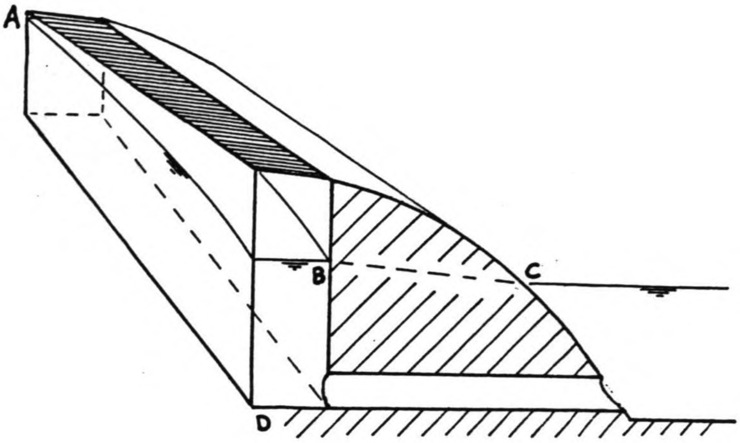


FIGURE 4.14

Hydraulic limits

Velocities in the trench must be sufficient to prevent the settling of too much sediment.
With an opening width between the grid bars of 10 mm the maximum diameter of cobbles in the trench would be D = 10 mm. As an acceptable limit a **minimum velocity** of **v = 1 m/s** is chosen to occur for at least 80 % of the length of the trench. This should keep the cobbles moving, according to Shields' criterion for movement [litt.12]:

$$(h * s)/(\Delta * D) > 0.02 \qquad \text{with } \begin{array}{l} h = 3.6 \text{ m.} \\ s = 0.10 \\ \Delta = 1.65 \\ D = 10 \text{ mm.} \end{array}$$

(These values are in accordance with the trench dimensions established below.)

The water surface profile can be computed with the dynamic equation for spacially varied flow with increasing discharge (figure 4.13):

$$dh/dx = \frac{s_o - 2\, q^2 * x/(g*A^2)}{1 - q^2 * x^2/(g*A^2*R)} \qquad \text{[litt.4]}$$

In Appendix A this computation is described in more detail.

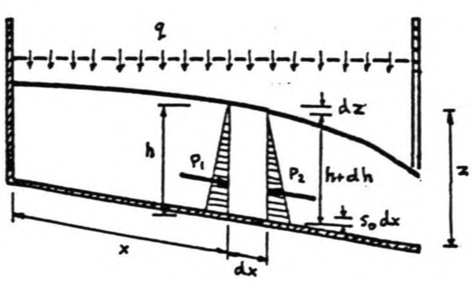


FIGURE 4.15

Spatially varied flow

There are two parameters that can be varied:

- the depth at the outlet (h_B), which is the control section when the flow is sub-critical,
- the slope of the trench bottom (s_o).

Since the weir should be is low as possible, the manipulation of the variables is aimed at:

- minimum z_A,
- minimum v = 1 m/s at x = 5 m.

The results of these computations is shown in figure 4.16.

The optimum appeared to be a **bottom slope of 1 : 10**, and a **water depth h_B = 3.1 m.** at the outlet.
The flow is sub-critical in every section but the last, the maximum velocity being v = 5.5 m/s., which is not too high for flow in a concrete channel.

The total **depth of the trench D_t = 5.7 m.** below the intake grid. The grid level must therefore be set at 734.5 + 5.7 = 740.2 m.. This implies **raising the weir with 2.5 m.**, since the grid level in the NEA design is only 737.7 m.EL.

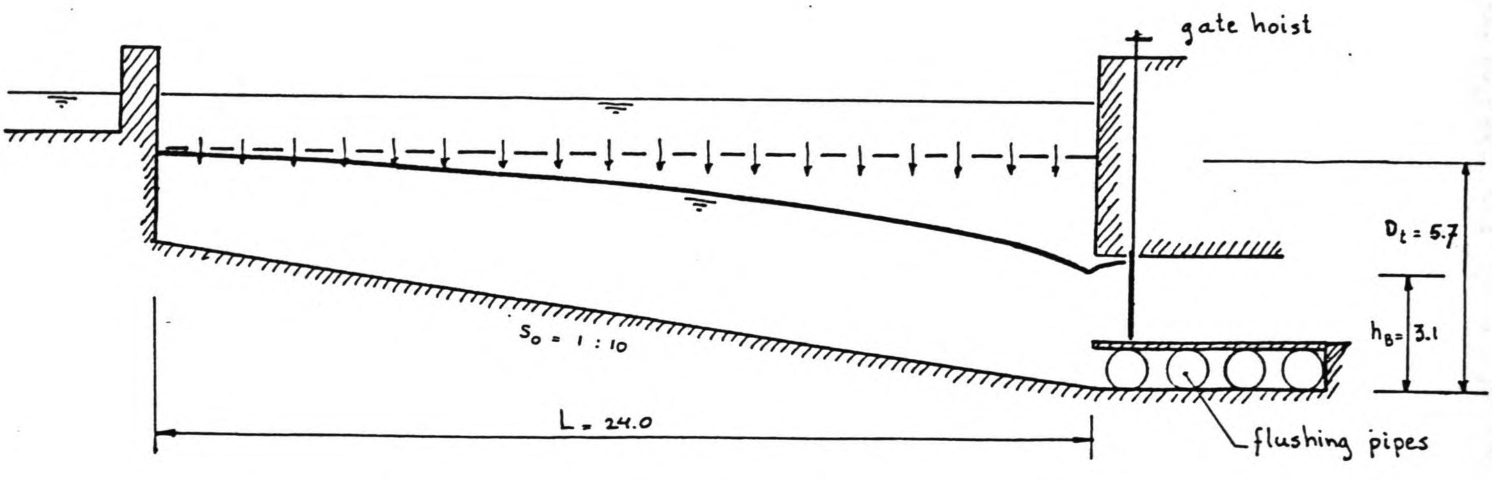


FIGURE 4.16 Flow profile in collecting trench

4.2.2.3. Dimensions of flushing pipes

The capacity of the flushing pipes should be at least equal to the intake capacity of the grid under the design flood conditions: **Q_F = 34 cumecs.**

The limiting conditions are:

- **tailwater level = 736.8 m. (see table 2.3)**
- minimum water level in the collecting trench when turbine discharge = 0.

The pressure head at the pipe entrance is given by the water depth in the collecting trench at full flow:

h_B = 3.1 m.

The available head drop therefore is (figure 4.17):

ΔH = (734.5 + 3.1) − 736.8
= 0.8 m.

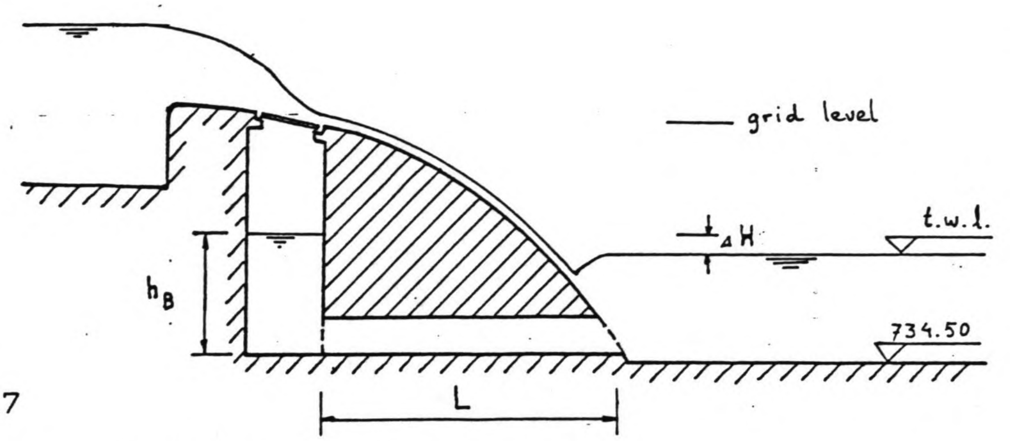


FIGURE 4.17

Cross section over flushing pipe

For **four** pipes of diameter **D = 1.0 m**. the **friction losses** can be computed with the Hazen-Williams formula:

$s = (v/(0.85 * c * R^{0.63}))^{1.85}$ (c = 130 for steel)
$v = Q_F/A$ = 10.8 m/s
so **s = 0.068**

With a pipe length L = 7.5 m. this gives: ΔH_F **= 0.51 m.**

The **entrance losses** are dependant on the shape of the pipe entrance (figure 4.18):

$\Delta H_e = \xi_i (v^2/2g)$

For slightly rounded edges the losses are:

$\Delta H_e = 0.05 * (10.8^2 / 2g) = 0.30$ m.

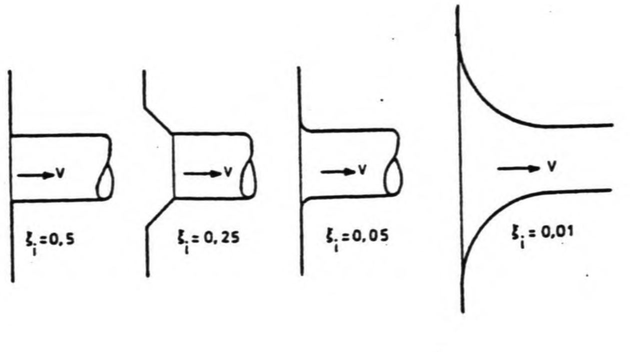


FIGURE 4.18

Coefficients of entrance losses

The total necessary head drop over the pipes therefore is:

$$\Delta H = 0.51 + 0.30 = 0.81 \text{ m.}$$

This is about the same as the available head drop, when the water level in the trench is minimal. There is a considerable safety margin however, since the water depth at the trench outlet can rise to h_B = 3.6 m. without endangering the fulfilment of the design criteria (see Appendix A, Table A.3).

4.3. CONCLUSION

The minimum width of the Tyrolian intake is b = 24 m. A bottom intake of this size cannot be kept free of sediment under pressure flow conditions. To maintain free surface flow flushing pipes must be included in the design, regulated by gates.
Analysis of capacities and dimensions of the structure shows that it cannot be built in a weir that creates only 2 m. head drop. The weir crest must be raised to 740.2 m.EL **which increases the total costs with about 40 %**.

For this reason the Tyrolian bottom intake is not a suitable type for the Jhimruk Khola. A frontal or lateral type of intake is more appropriate. Since only a small part of the river discharge is diverted during the monsoon period (when sediment transport is a problem), the frontal intake has no significant advantage over the lateral type. Its disadvantages have been pointed out in section 4.1.3., which leads to the conclusion that **the intake in the Jhimruk Khola should be of the lateral type.**

5. ALTERNATIVE INTAKE AND DESILTING ARRANGEMENT

5.1. LOCATION OF DESILTING BASIN

A desilting basin or sand trap is a sediment ejection structure, that removes (most of) the particles bigger than a specified D_{max} from the water that has entered the intake. For hydro-electric projects D_{max} = 0.25 mm. usually. The basic principle of a sand trap is lowering the velocity of the water in a wide basin, to enable the sand to settle, and extracting the water from the clean upper layer at the end of the basin.

Depending on the local conditions the removal of the sand can be done periodically or continuously. The removal mechanism can be either mechanical or hydraulical. For the Jhimruk Hydro-electric Project the only feasible solution is continuous flushing, because of the high sediment load during the monsoon (see Appendix C).

For the location of the sand trap it is necessary to make an estimate of the hydraulic demands for the basin site. Roughly it can be stated that for this combination of river and intake some 10 - 15 % of the turbine discharge is needed for flushing, and the water level difference between basin and river should be at least 3 m. during the design flood (figure 5.1).

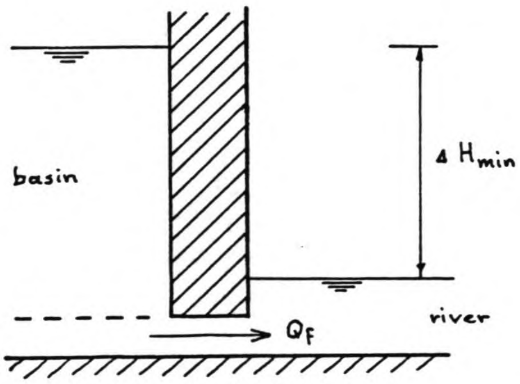


FIGURE 5.1

Head drop over flushing system

The sand trap location should not only fulfil the water level demand, but the extra flushing discharge and heavy sediment load should also pose no problems. For these reasons the desilting basin should be located at the bank of the Jhimruk Khola itself, where it will be no problem to dispose of the sediment-laden flushing water.

The minimum head drop of 3 m. poses a problem, since the weir only creates a drop of 2 m.. Usually this is solved by locating the basin a sufficient distance downstream of the intake. The slope of the canal between the intake and the basin can be smaller than the river slope, and thus an increasing head drop is achieved with increasing distance.

The total head drop ΔH_t at a distance l downstream of the weir is dependant on the weir height ΔH_w, the river slope s_r and the canal slope s_c (figure 5.2):

$$\Delta H_t = \Delta H_w + l * (s_r - s_c)$$

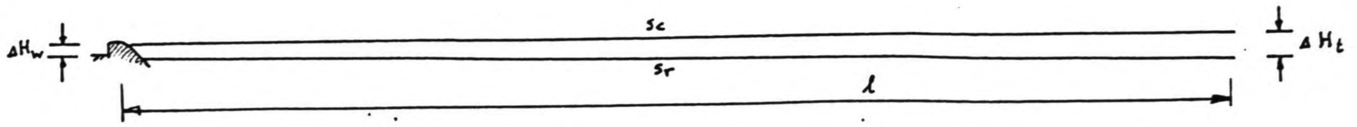


FIGURE 5.2 Development of head drop
(different horizontal and vertical scale)

So the mimimum distance is:

$$l = (\Delta H_t - \Delta H_w) / (s_r - s_c)$$

with H_t = 3 m.
H_w = 2 m.
s_r = 0.6 %
s_c = 0.1 %

this gives:

l = 200 m.

The slopes on the right bank are too steep (40°) and unstable however to allow construction of a canal. The alternatives are:

- a canal in the river bed itself
- a tunnel in the river bank rock.

The first would be exposed to both the river floods and the occasional landslide coming down the slopes. Adequate protection would be very costly. The second option also is very expensive.

There is one area where the water level is lower than in the corresponding section of the river: **right behind the gates in the sluiceway**, which is located close to the intake to keep the river bed at sufficient depth there. In the monsoon, when flushing of the sand trap is necessary, these gates are open. Behind the (radial) gates there is an area of supercritical flow, with small water depth and low pressure head (figure 5.3).

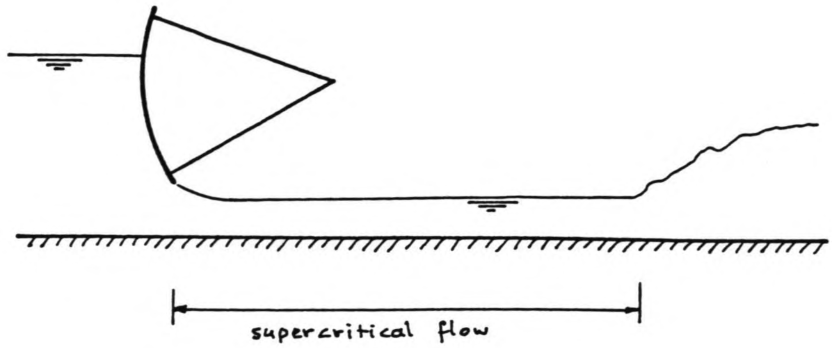


FIGURE 5.3 Flow behind a radial gate

If the flushing pipes of the desilting basin come out into this area, this could give enough extra head for adequate flushing. A quantative analysis is given in chapter 6., to determine the optimum dimensions, the sensitivity to operation of the gates, and the ability to function under the design flood conditions.

In order to be able to flush into the supercritical flow, it is necessary to locate the desilting basin as close to the sluiceway gates as possible. The gate should also be close to the intake. Intake and desilting basin are therefore designed as one box-like structure, as compact as possible, with the sluiceway alongside it (figure 5.4).

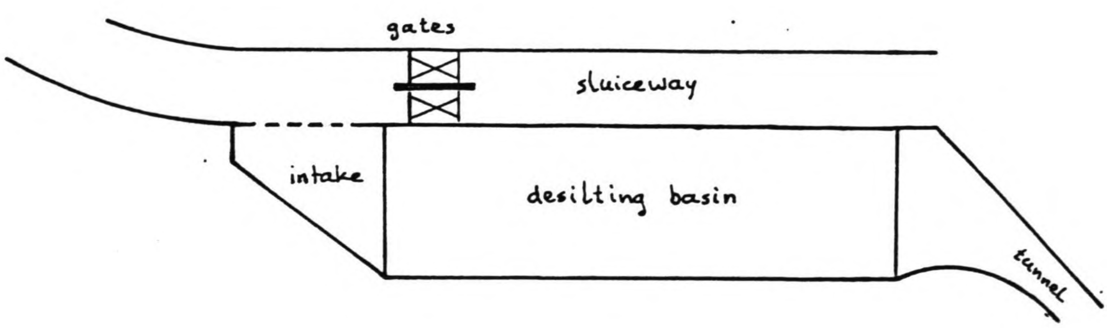


FIGURE 5.4 Intake and desilting arrangement

5.2. ALIGNMENT OF SLUICEWAY

The sluiceway has three functions, which generally should be fulfilled simultaneously:

- to keep the approach channel to the intake at sufficient depth
- to lower the water level for flushing purposes
- to transport the sediment from the sand trap back into the river.

Width and depth are based on the determining combinations of river conditions and turbine operation (calculations are given in chapter 6.).

The alignment is based on qualitative considerations. It consists of (drawing 2):

- a deepened approach channel in the river bed
- an artificial bend before the intake
- a straight section with the gates
- the end protruding at an angle back into the river
- the location avoiding interference from the gully just downstream of the weir.

The **approach channel** will have an equilibrium that is constantly shifting due to changing river conditions. During monsoon there is a danger of silting up, which will have to be countered by operation of the gates.

The **bend** is introduced to create a spiral flow (see section 4.1.2.) which should keep the sluiceway in front of the intake at depth. The section in front of the gates is **straight**, to minimise turbulence and vibrations.

A horizontal contraction is created in the section behind the gates, to fix the position of the hydraulic jump (see section 6.3.3.2).

The **end** of the sluiceway points back into the river, to facilitate the removal of sand and thereby avoid blocking. The **distance** from the bank should keep the sluiceway safe from blocking by landslides etc..

Finally the **location** is fixed by the position of the tunnel, which should start in the relatively firm rock just downstream of the gully coming down the right bank.

The radius of the bend (R = 30 m.) and the angles of inflow and outflow with the river axis (45° and 30° resp.) are based on some rough guidelines for design and descriptions of similar structures in other projects [litt.1 and 13]. They should be checked in model tests for this specific case.

5.3. POSITION OF INTAKE WINDOWS

The lateral intake has the form of a box with "windows" to let the water in. Their size is determined in chapter 6. Their position in relation to the sluiceway alignment is chosen to make maximum use of the spiral current in the curved section. Experiments and experience has shown that the maximum scour at a bend occurs at a distance of about two times the width of the channel from the point where the axis of the oncoming flow meets the outer bank (figure 5.5).

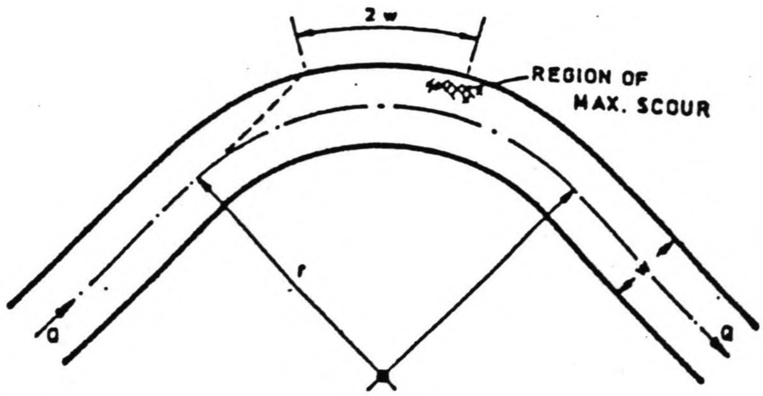


FIGURE 5.5 Scour in a river bend

Locating the intake windows in this area should minimise sediment intrusion and keep the sluiceway in front of it at sufficient depth to avoid the bed load.

The windows should be placed at sufficient height above the sluiceway bottom to allow the bed load to pass underneath it to the gates and back into the river. Quantitive design methods for determining the minimum height are not available. A rough guideline is: p = 1/3 h [litt.1] (figure 5.6).

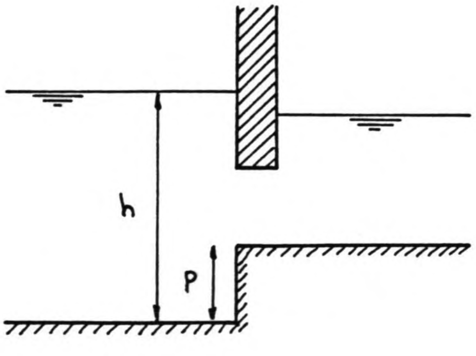


FIGURE 5.6 Intake sill height

The water level at the intake will probably vary between 737.65 m.EL and 737.85 m.EL during the monsoon period (see table 6.1).
On this basis the **level of the intake sill** is chosen to be **736.60 m.EL**, and the sluiceway bottom level EL. 736.0 m..
so h ≈ 1.75 m. and p = 0.60 m.
A higher level of the intake windows would imply increasing their width to attain their minimum capacity. A lower level of the sluiceway bottom would make it more difficult to keep open, and more costly to build.

5.4. FLUSHING SYSTEM

A desilting basin with continuous flushing should have troughs in its bottom to catch the settling sand and transport it back to the river. The usual arrangement is a **longitudinal** channel with the basin bottom sloping towards it (figure 5.7). The trough must be partly covered to reduce the flushing discharge and to assure sufficient flushing velocity over its total length.

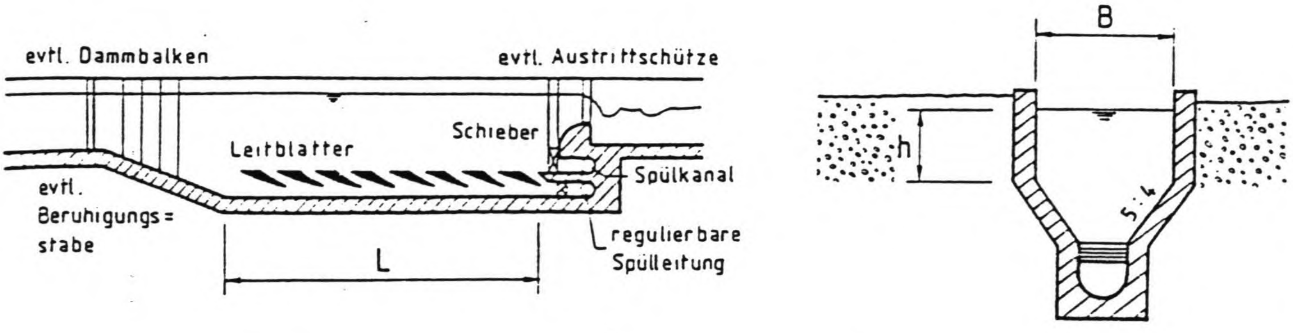


FIGURE 5.7 Longitudinal flushing channel

The alternative is a number of **lateral** channels, that spill their discharge sideways out of the desilting basin (figure 5.8).

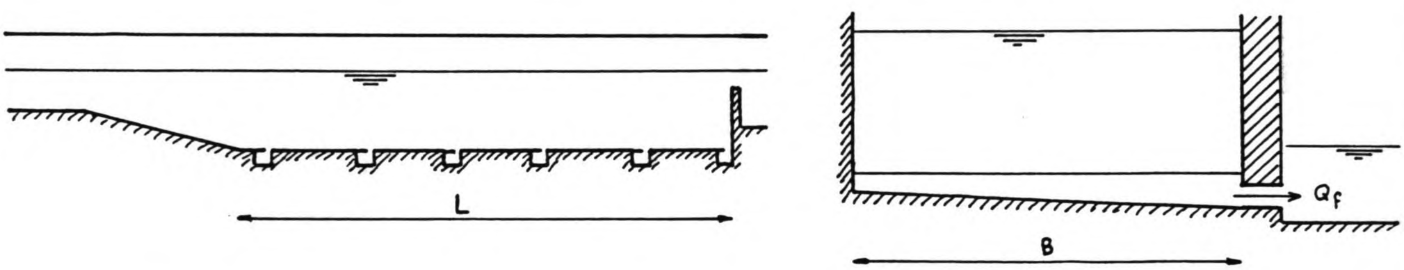


FIGURE 5.8 Lateral flushing system

The advantage is the much smaller length of the channels, which makes it easier to keep them open. This is especially important where there is only a minimum head available for flushing, as is the case for the Jhimruk Khola intake. A disadvantage is the need for more gates, since every flushing pipe outlet must be closed during the dry season to avoid water losses.

The flushing pipes will have to come out into the area of supercritical flow behind the gates. Since the length of this area is estimated to be about half the length of the desilting basin it is necessary to have some longer channels in the second half of the basin. Figure 5.9 shows a schematic plan of the basin with flushing channel arrangement.

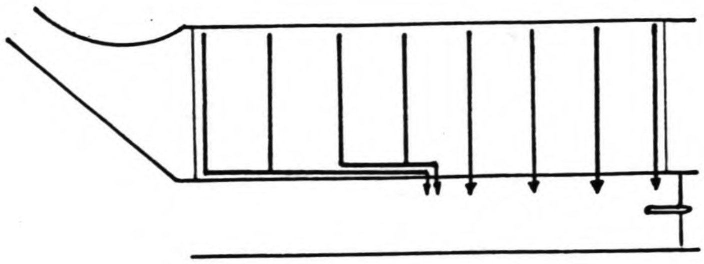


FIGURE 5.9

Flushing channel plan (schematic)

5.5. REGULATION OF DISCHARGES

5.5.1. Water balance

The water balance of the intake and desilting system can be described in several equations:

- $Q_r = Q_w + Q_s + \Delta R$
- $Q_s = Q_g + Q_i$
- $Q_i = Q_f + Q_t$

Q_r = river discharge
Q_w = discharge over weir
Q_s = sluiceway discharge
ΔR = increase of storage
Q_g = gate discharge
Q_i = intake discharge
Q_f = flushing discharge
Q_t = turbine discharge

or combined:

- $Q_r = Q_w + Q_g + Q_f + Q_t + \Delta R$ (in cumecs)

The limiting conditions are given by Q_r and Q_t.

5.5.2. Flow over weir

The weir discharge can be regulated by several kinds of structures that make the weir height adjustable. Most are very expensive, especially for a 250 m. long weir. The most cost effective solution is the use of flashboards or some other type of temporary wooden structure. The costs of this structure must be justified by the extra generating capacity that can be achieved with the created storage capacity. Seasonal storage is impossible with this type and size of project, so the reservoir is meant to balance the variations in the daily demand (figure 5.10).

An economic analysis by the NEA has shown the optimum generating capacity to be 10.5 MW. The NEA report states that a reservoir of 82,000 m^3 capacity is necessary to guarantee the four hours of peaking power during the dry season. However, if the load curve given in figure 2.8 is used to determine the reservoir size the result is a minimum of only 50,000 m^3 (see figure 5.10). This 24-hour operation is only possible with a minimum river discharge of 1.73 cumecs. When the river discharge is at its 1-day

90 % dependable minimum in May (Q_r = 0.92 cumecs) the turbines must be shut down for ± 20 hours. In this case a reservoir of ± 70,000 m^3 should be sufficient for the four peak hours. The proposed 82,000 m^3 thus gives a 17 % margin for higher loads or lower river discharges.

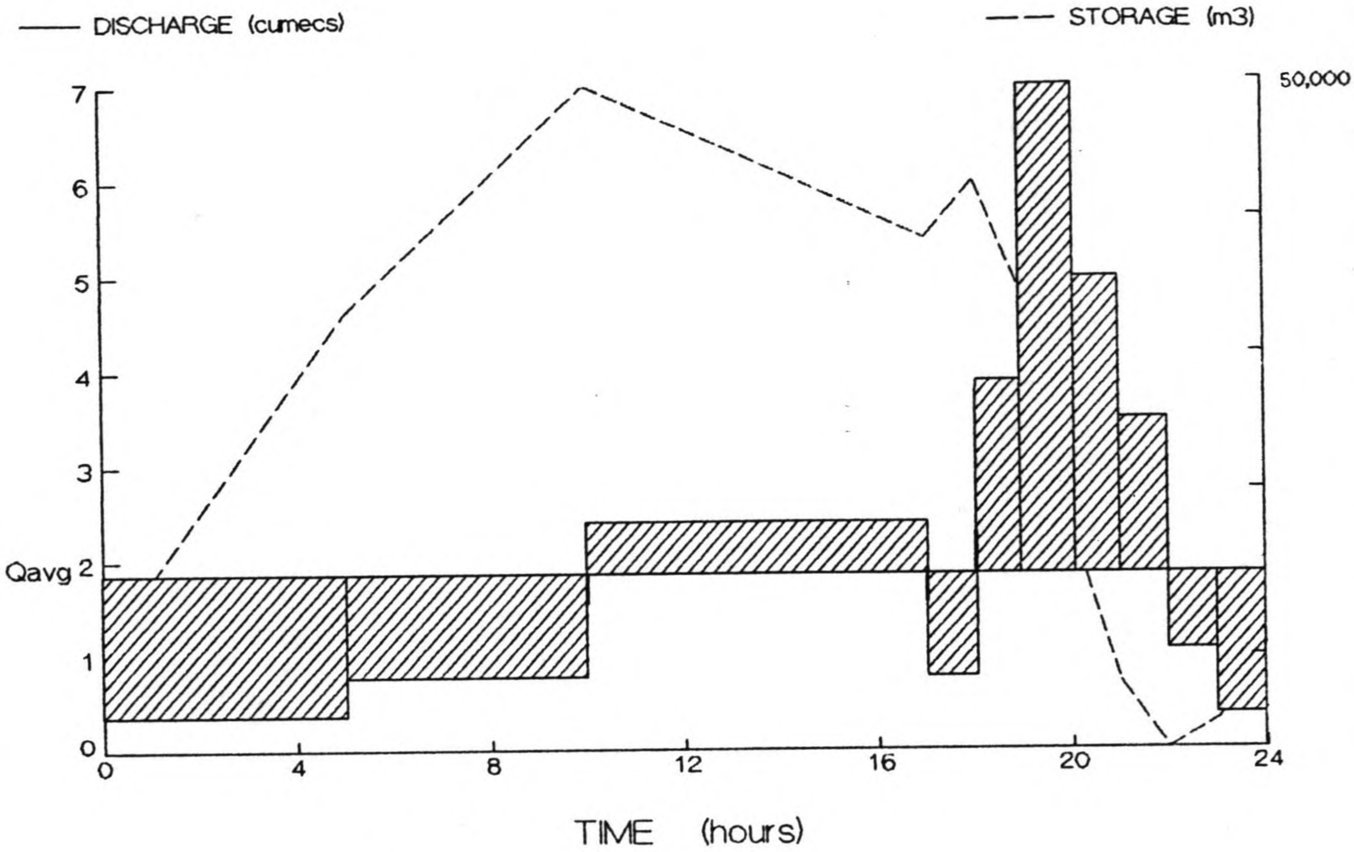


FIGURE 5.10 Turbine discharge and reservoir storage

With the proposed flashboards it is not possible to regulate the flow over the weir in a variable way: they are either up or down. They are installed at the beginning of the dry season, and removed at the beginning of the monsoon, as soon as the river discharge is reliable enough. Without the flashboards $\Delta R = 0$. With flashboards up $Q_w = 0$ as long as the storage R has not reached its maximum; when it does the water will spill over the boards and $\Delta R = 0$ again.

5.5.3. Sluiceway gate discharge

The flow through the sluiceway is regulated by a set of gates. A **radial gate** type is chosen because of

- less leakage at the gate sides
- easier lowering in fast flowing water

in comparison to the conventional sliding gates. The gates will be closed during the dry season and opened at the beginning of the monsoon.

The main function of the flow through the sluiceway is to keep the intake area free of sediment build up, and the approach channel at sufficient depth. The second function is to create a sufficient length of supercritical flow behind the gates. Operation of the gates should aim at these two functions simultaneously. Yet the system should not be too dependant on accurate operation at all times. The **sensitivity** of the system will be checked in chapter 6., to see if the main parameters (velocities; depths; position and length of hydraulic jump) can be kept within acceptable limits with simple operational procedures.

5.5.4. Intake discharge

The discharge entering the intake through the windows is dependant on the difference in water level inside and outside the intake (figure 5.11).

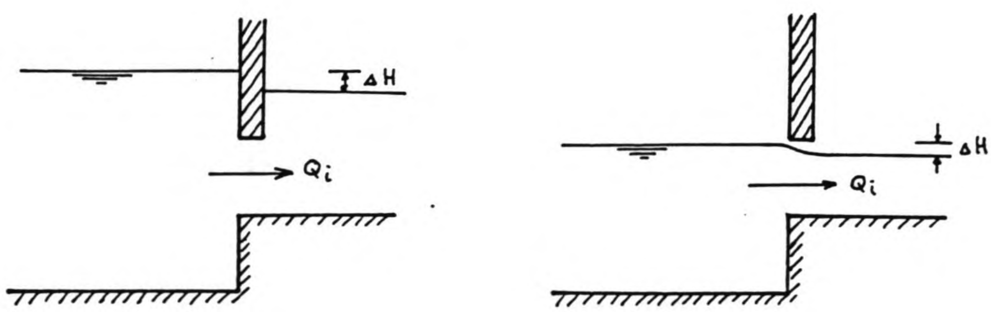


FIGURE 5.11 Discharge through intake windows

No seperate regulator is envisaged at the entrance. If the turbine discharge is zero, and the flushing system of the desilting basin is closed, the inside water level will rise to the level outside, and the inflow will cease. The intake is **self-regulating**: $Q_i = Q_t + Q_f$ at all times.

It should be possible to close the windows completely however, to be able to drain the basin for repairs etc.. For this purpose the use of stop logs is the most cost effective.

5.5.5. Flushing discharge

Each flushing pipe must have a gate to close it off during the dry season. In the monsoon they can be kept open all the time, since there is ample water available for flushing. No accurate regulating devise and operational procedures are required here; simple small sliding gates will do.

5.5.6. Turbine discharge

The flow through the turbines is directly linked to the power demand, and is regulated in the powerhouse itself. Due to the inertia of the water flowing through the tunnel and the penstock pressure waves (surge) will occur when the turbine discharge changes abruptly. The surge tank between the tunnel and the penstock will cut the edge off these waves. There is no necessity therefore to have a regulating devise at the entrance of the tunnel.

It is necessary however to be able to close the entrance and drain the tunnel. Because this should be possible on short notice in an emergency stop logs are not a feasible solution, and a **sliding gate** (with rack and pinion) is opted for.

6. DIMENSIONS

6.1. INTAKE

6.1.1. Intake chamber

The intake chamber is shown in figure 6.1. Here the water flowing through the intake windows is collected, and led into the desilting basin.

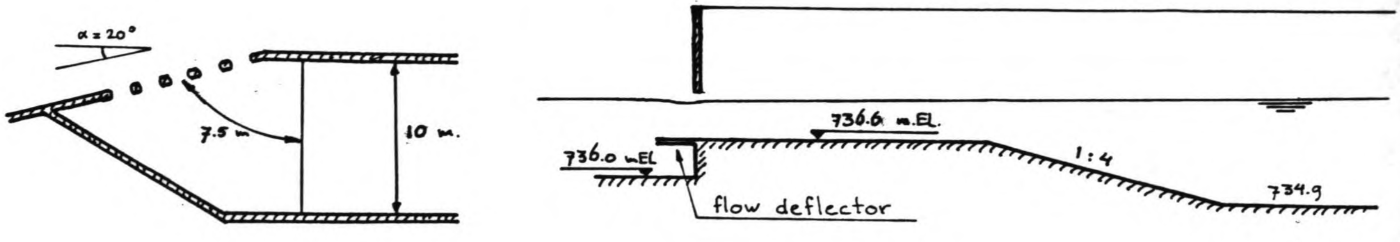


FIGURE 6.1 Intake chamber plan & section

The level of the concrete floor is the same as that of the window sill (736.60 m.EL.). Lowering it to the level of the desilting basin floor would reduce the velocities in the chamber, and therefore result in the settlement of sand directly behind the intake windows. This is only acceptable however in the desilting basin itself, where the sand can be removed.
The flow deflector in front of the intake windows should prevent the intrusion of bed load. Its length is the same as the sill height: 0.60 m.
The average width of the intake chamber is about 8 m., the average length about 7.5 m.
The chamber ends with a transition, where the floor level drops to that of the desilting basin. To minimise energy losses the slope of this transition is 1 : 4.

6.1.2. Intake windows

The intake discharge flows through windows in the wall of the intake chamber. These are covered with coarse trash racks, with bar spacing a = 100 mm. and diameter D = 20 mm. The window frames have grooves for inserting stoplogs, to enable their closure and the draining of the intake chamber and desilting basin (figure 6.2.).

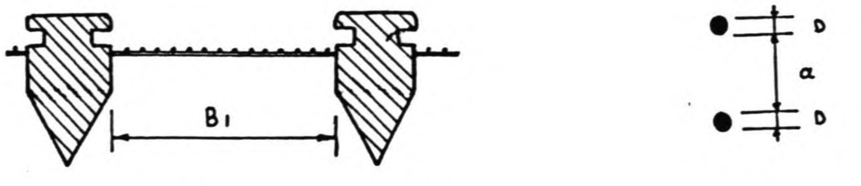


FIGURE 6.2 Intake window and trash rack

The minimum dimensions of these windows can be determined for the limiting conditions: maximum turbine and flushing discharge, and minimum river discharge. For the river the 90 % dependable flow is used, and the flushing discharge is estimated to be 30 % of the turbine discharge. Two cases can be distinguished:

- monsoon:	$Q_i = Q_t + Q_f$	= 9.0	cumecs	
	$Q_r = Q_{90,july}$	= 30	cumecs	
	BWL =	= 738.1	m.EL.	(table 2.3)
- dry season:	$Q_i = Q_t$	= 6.9	cumecs	
	$Q_r = Q_{90,may}$	= 0.9	cumecs	
	BWL =	= 738.0	m.EL.	

For the water level in the dry season the minimum has been chosen (reservoir practically empty).

The minimum intake width B can be computed assuming critical flow in the section of contraction of the trash rack. The intake chamber will act as a broad-crested weir, with its control section being the contraction. The discharge formula is:

$$Q_i = \mu * c_d * 2/3 * \sqrt{(2/3\ g)} * B * H^{1.5}$$

In case I the velocity head of the approach flow is considerable, because the sluiceway is being flushed:

$v^2/2g = 0.29$ m. (see table 6.1)

Only part of this kinetic energy can be recovered as potential energy in the intake, depending on the angle between the intake windows and the approach flow direction. In this case $\alpha = 20°$ (see figure 6.1), so

$$v_i^2/2g = \sin^2\alpha * 0.29 = 0.03 \text{ m.}$$

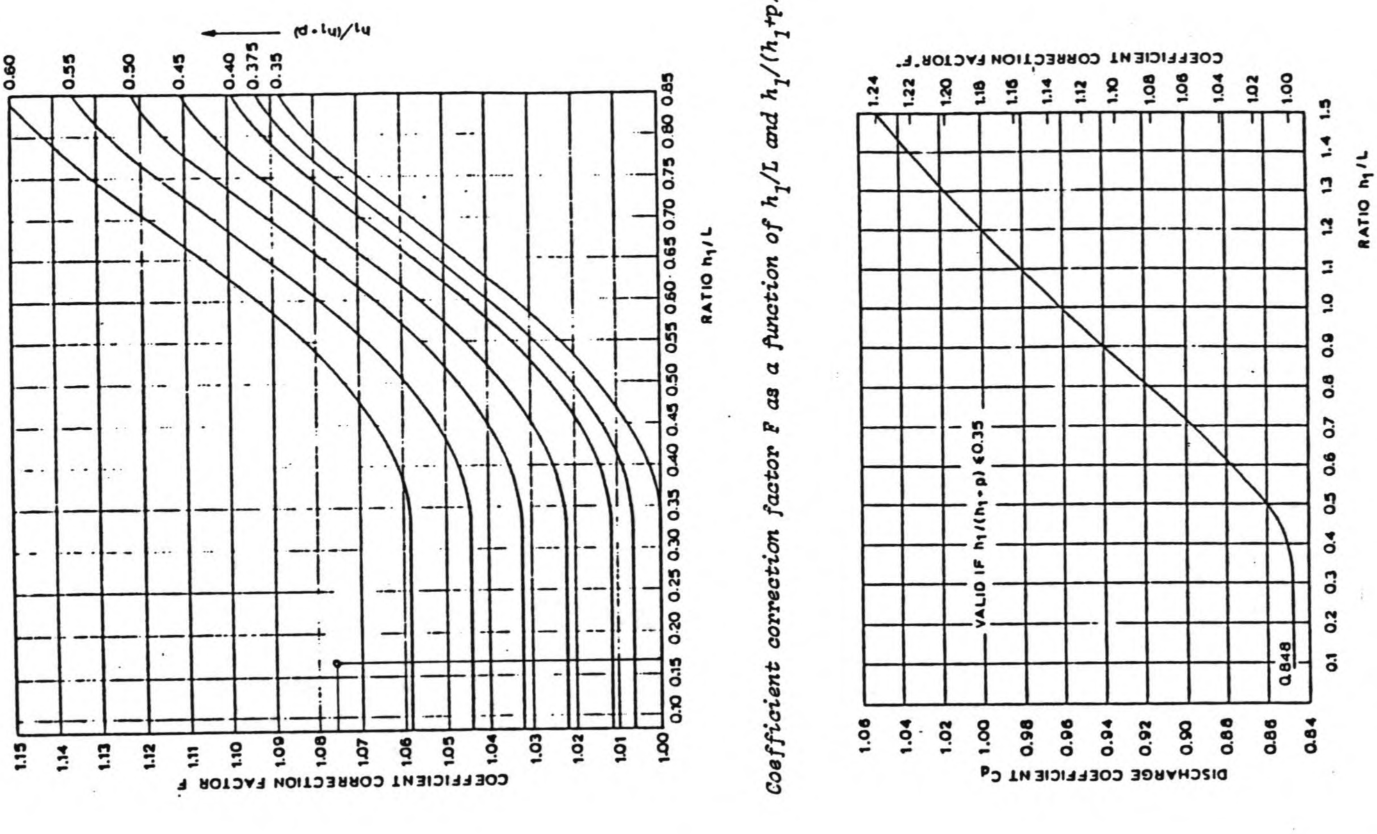
The energy head $H = h + v^2/2g$ therefore is:

case I : $H = (737.75-736.60) + 0.03 = 1.18$ m.
case II: $H = 738.0 - 736.60 = 1.40$ m.

The discharge coefficient c_d is dependant on the ratios H/L and H/(H+p), with L = "weir" crest length and p = sill height (figures 6.1 and 6.3).
In this case

$H \approx 1.3$ m. $L \approx 7.5$ m. $p = 0.60$ m.

so $c_d = f * c_{d,min} = 1.075 * 0.848 = 0.915$.



Coefficient correction factor F as a function of h_1/L and $h_1/(h_1+p)$

C_d-values and F-values as a function of h_1/L, provided that $h_1/(h_1 + p) \leq 0.35$.

FIGURE 6.3 Discharge coefficients [source: litt.3]

For round bars μ = 0.85 , so

$q = 1.367\ H^{1.5}$ and $B_{min} = Q_{max}/q$

Case I: H = 1.18 m.
Q_{max} = 9.0 cumecs
so B_{min} = 5.07 m.

Case II: H = 1.40 m.
Q_{max} = 6.9 cumecs
so B_{min} = 3.40 m.

Case I (monsoon) is the limiting combination of conditions. If we take ± 50 % safety margin (for irregular shapes, trash caught in the rack etc.): **B = 7.5 m.**

By trial and error (using a clearness factor of 0.90 for the rack) the water depth at the control section can then be found: h = 1.15 m., and the velocity v = 1.2 m/s.

The rack losses can now be determined with Kirschmer's formula [litt.11](figure 6.4.):

$$\Delta H = \emptyset * (D/a)^{1.33} * \sin\alpha * (v^2/2g)$$

with $\emptyset$ = 1.79 (round bars)
D = 20 mm.
a = 100 mm.
α = 90 deg.
v = 1.2 m/s

this gives: ΔH = 0.02 m.

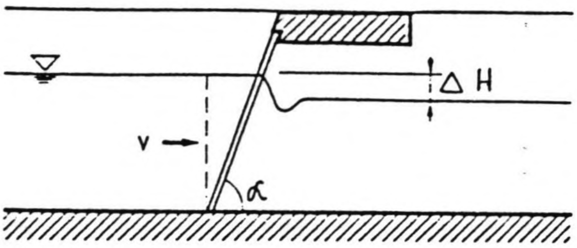


FIGURE 6.4

Flow through a grid

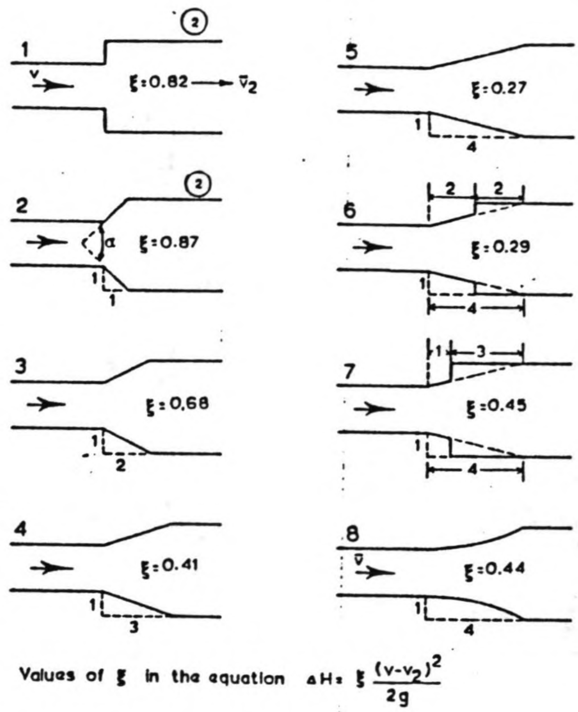


FIGURE 6.5

Expansion losses

(source: litt.3)

Some head losses can also be expected in the transition to the desilting basin. For a 1 : 4 expansion these are given by (figure 6.5.):

$$\Delta H_e = 0.27\ (\Delta v)^2/2g$$

With Δv = (1.2-0.3) = 0.9 m/s (see section 6.2.1.1.) this gives:

$$\Delta H_e = 0.01 \text{ m.}$$

Friction losses in the intake chamber plus transition can be estimated with Manning's formula [litt.4]:

$$v = M * R^{0.67} * \sqrt{s_f}$$
or $$\Delta H_f = L * (v / M * R^{0.67})^2$$

With $M = 60$ (unfinished concrete)
$R = 0.9$ m. (average)
$v = 0.8$ m/s (average)
$L = 7.5$ m. (average)
this gives:

$$\Delta H_f = 0.002 \text{ m.}$$

Total head losses over the intake can therefore be estimated at **$\Sigma\Delta H = 0.03$ m.**, which confirms that the arrangement results in a minimum loss of energy.

6.2. DESILTING BASIN

6.2.1. Main dimensions

The main dimensions of the basin are width B, length L and depth h (or, more accurately, the floor level FL) (figure 6.6.).

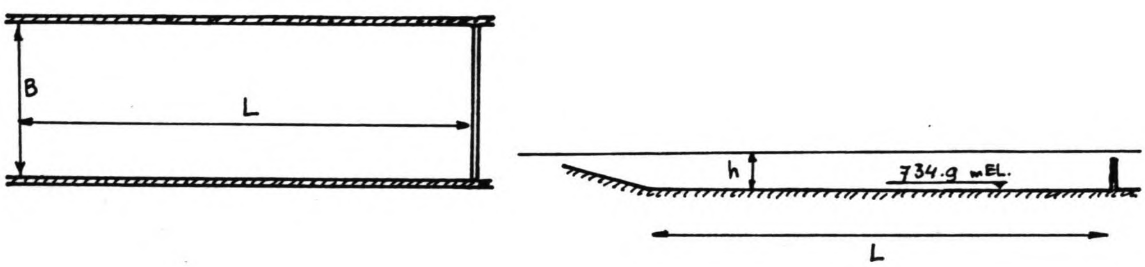


FIGURE 6.6 Desilting basin main dimensions

Hydraulically the average velovity $v = Q_{avg}/(B * h)$ is decisive. The **design criterion** is the catching of every grain (or more than 95 % of the grains) of sand with $D > D_{max}$. For hydro power projects the acceptable **$D_{max} = 0.25$ mm.**

6.2.1.1 Basin floor level

The level of the basin floor is limited by the level of the sluiceway floor, since the sand in the basin must be flushed into the sluiceway. The level of the sluiceway is set at 734.0 m.EL (see section 6.3.3.), and the **level of the basin floor is 734.9 m.**EL (which leaves 0.9 m. for the flushing system underneath the floor).

6.2.1.2. Basin width

The minimum depth during the monsoon season is (case I):

$$h = H - \Delta H - FL - v^2/2g$$
$$= (736.60 + 1.18) - 0.03 - 734.9 - 0.3^2/19.6$$
$$= 2.84 \text{ m.}$$

since the average velocity in a sand trap is usually chosen to be **v = 0.3 m/s.**

With Q_{avg} = (9.0 + 6.9)/2 = 7.95 cumecs, this gives:

$$B_{min} = Q/(v * h) = 9.33 \text{ m.}$$
and $A_{min} = B_{min} * h = 26.5 \text{ m}^2$

Because the cross section is not quite rectangular (figure 6.10) we take **B = 10.0 m.**

6.2.1.3. Basin length

The length can be determined if we know the settling velocity of the smallest particle that must be trapped. From figure 6.7. it is easily seen that

$$v/w = L/h$$

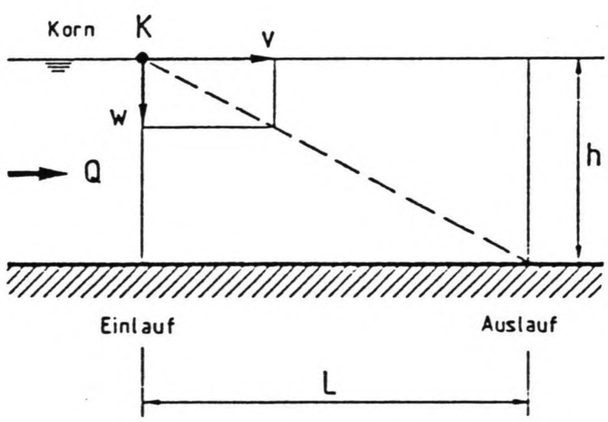


FIGURE 6.7

Settling trajectory

The settling velocity in still water is given by Stokes as [litt.11]:

$$w_o = \Delta * g/(18 * \nu)\ D^2$$

with $\Delta = \rho_s - \rho_w$
ν = kinematic viscosity
D = grain diameter

This formula is valid for laminar flow around the grain: if $Re_g = w_o * D/\nu < 1$.

For turbulent flow (Re_g > 2300) the formula of Prandtl must be used [litt.11]:

$$w_o = \sqrt{(\Delta * 4/3 * g * D/c)}$$

with c = friction coefficient (= 0.5 for round grains).

For 1 < Re_g < 2300 there is a linear transition between these formula's, if plotted on double logarithmic scales.

In this case: $\Delta = 1.65$ (sand)
$\nu = 1.32 * 10^{-6}$ m^2/s (water of 10 °C)
$D = 0.25$ mm.

which gives:

$$w_o = 0.04 \text{ m/s} \quad \text{using Stokes.}$$

Check: Re_g = 7.6 , which confirms that the flow around the grain is almost laminar, and the use of Stokes' formula justified.

The real settling velocity is lower, because of the turbulent flow in the basin. This difference can be estimated with:

$$w' = 0.04 \; v$$

with v = average velocity in the sand trap.

So $w = w_o - w'$
$= 0.04 - 0.04 * 0.3 = 0.028$ m/s.

The minimum length can now be determined:

$$L = h * v/w = 30.4 \text{ m.}$$

For practical purposes we choose **L = 33.0 m.** , which gives a safety margin of about 7.5 % overall.

Summarising: B = 10.0 m.
L = 33.0 m.
FL = 734.9 m.EL.

6.2.1.4 Maximum basin discharge

The maximum acceptable discharge can now be determined for the maximum flood level under which the system is operated:

Q_r = 500 cumecs
sluiceway WL = 737.75 m.EL (table 6.1)
sluiceway v = 4.5 m/s

The basin water level then is:

$$WL = 737.75 + (\sin \alpha * v)^2/2g$$
$$= 737.87 \text{ m.EL}$$

and the water depth:

$$h = WL - FL$$
$$= (737.87 - 734.90) = 2.97 \text{ m.}$$

Using the same safety factor and thus the same length $L = 30.4$ m., the acceptable average velocity is given by:

$$v/w = L/h = 10.24$$

and $w = 0.04 - 0.04 * v$

so $v = (10.24 * 0.04)/(1 + 10.24 * 0.04)$
$= 0.29$ m/s

The maximum average discharge therefore is:

$$Q = v * B * h$$
$$= 0.29 * 9.5 * 2.97$$
$$= 8.2 \text{ cumecs}$$

With $Q_t = 6.9$ cumecs the **maximum flushing discharge under these conditions** therefore is:

$$Q_f = Q_{avg} - Q_t/2$$
$$= 8.2 - 6.9/2$$
$$= \mathbf{4.7 \text{ cumecs}}$$

This shows that there is ample discharge available for flushing, without endangering the efficiency of the desilting basin.

6.2.2. Flushing channels

The desilting basin is equiped with lateral flushing channels in its floor, that catch the sediment and transport it continuously to the sluiceway. Figure 6.8 shows such a channel in longitudinal and cross sections. The channels are concrete trenches in the floor, covered with tiles that keep a small slot open. The a-symmetrical position of the slot creates a spiral flow in the channel that increases its sediment transport capability. The tiles can be removed to allow cleaning, and shifted to adjust the slot width according to experience.

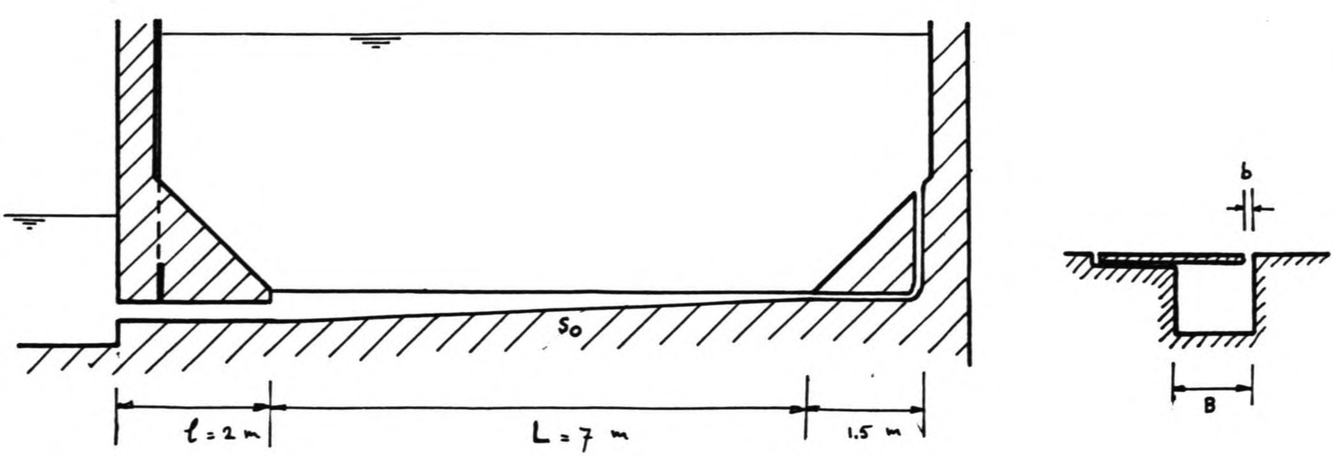


FIGURE 6.8 Flushing channel sections

The edges of the basin are "cut off" by 45° slopes, in order to shorten the channel length, and thus make it easier to keep it open without using too much water for flushing.

At the "upstream" end an initial discharge Q_o is provided from the cleaner layers of the basin flow, in order to keep that end from silting up. This is done by incorporating a small HDP (High Density Polyethylene) pipe in the sloping part of the basin.

The channel is connected to the sluiceway by a concrete pipe, that can be closed with a small gate. The length of these pipes is l = 2 m., the length of the channels is L = 7 m.

The channels in the last half of the basin are interconnected by a longitudinal collector (figure 6.9). For these channels the extra head loss of this collector is acceptable, since they transport only the smallest particles that settle in the last part of the basin, and the velocities can therefore be lower than in the other channels.

A number of eight channels is envisaged, with a distance of 4 m. between them. Small sand dunes will develop between the channels, but that is no problem as long as they don't grow high enough to have significant impact on the average velocities in the desilting basin.

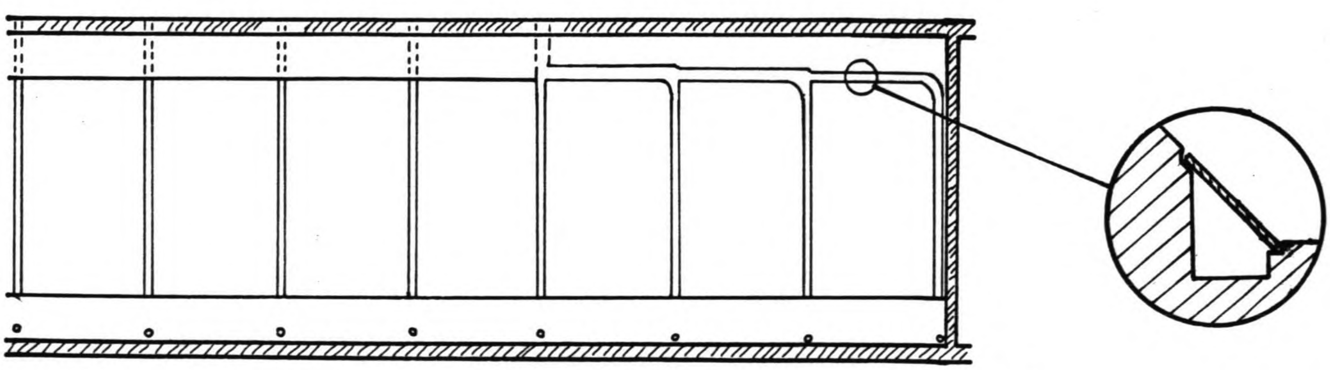


FIGURE 6.9 Flushing channel arrangement

The channel dimensions that have to be decided on are:

- channel width B
- slot width b
- initial depth h_o
- bottom slope s_o

The design criteria are:

- sufficient velocity over the whole length to prevent clogging of the channels. For this reason v = 1 m/s is taken as a minimum.
- sufficient discharge to flush all of the sediment entering the intake. The estimated minimum is: 15 % of the turbine discharge (see Appendix C).
- limited flushing discharge to avoid the need to enlarge the desilting basin. As a maximum 30 % of the turbine discharge is chosen, under Q_r = 30 cumecs (minimum in monsoon period). This amounts to 0.3 * 6.9 = 2.1 cumecs, or 0.26 cumecs per channel.
- sufficient width of the slot to avoid clogging. As a minimum b = 12 mm. is chosen, because gravels upto D = 10 mm are expected to enter the intake. The first channel could have a wider slot, and act as a gravel trap.

The flow in the channels can be computed with the aid of the **Bernouilli** equation, the **continuity** principle and the **momentum** equation [litt.4]. In Appendix B this is described in more detail.

In figure 6.10 the results of these computations are shown for the channel of figure 6.8 under the minimum water level conditions. It shows that the design criteria can be met when the dimensions are:

B = 0.20 m.
b = 0.012 m.
h_o = 0.10 m.
s_o = 0.03

and the water level in the sluiceway is 734.60 m.EL. or less (level of pipe outlet into the sluiceway).

The computations presented in Appendix B also show that the system is not too sensitive to the operation of the sluiceway gates.
When the hydraulic jump in the sluiceway is drowned the water level will be that of the tailwater. Even then only about 1 m. of the channel length runs the risk of clogging (see table B.3).
The same occurs under flood conditions (see table B.5).

It can therefore be **concluded** that under the design conditions ($Q_{r,min}$ = 30 and $Q_{r,max}$ = 500 cumecs) the sluiceway gates must be operated in such a way that the **water levels** in the area of super-critical flow are **below 735.0** and **735.8 m.EL.** respectively.
However, abberations are quite acceptable for short periods.
These maximum levels are in accordance with the necessary operation of the gates for the transport of sediment in the sluiceway itself (see section 6.3.3.2).

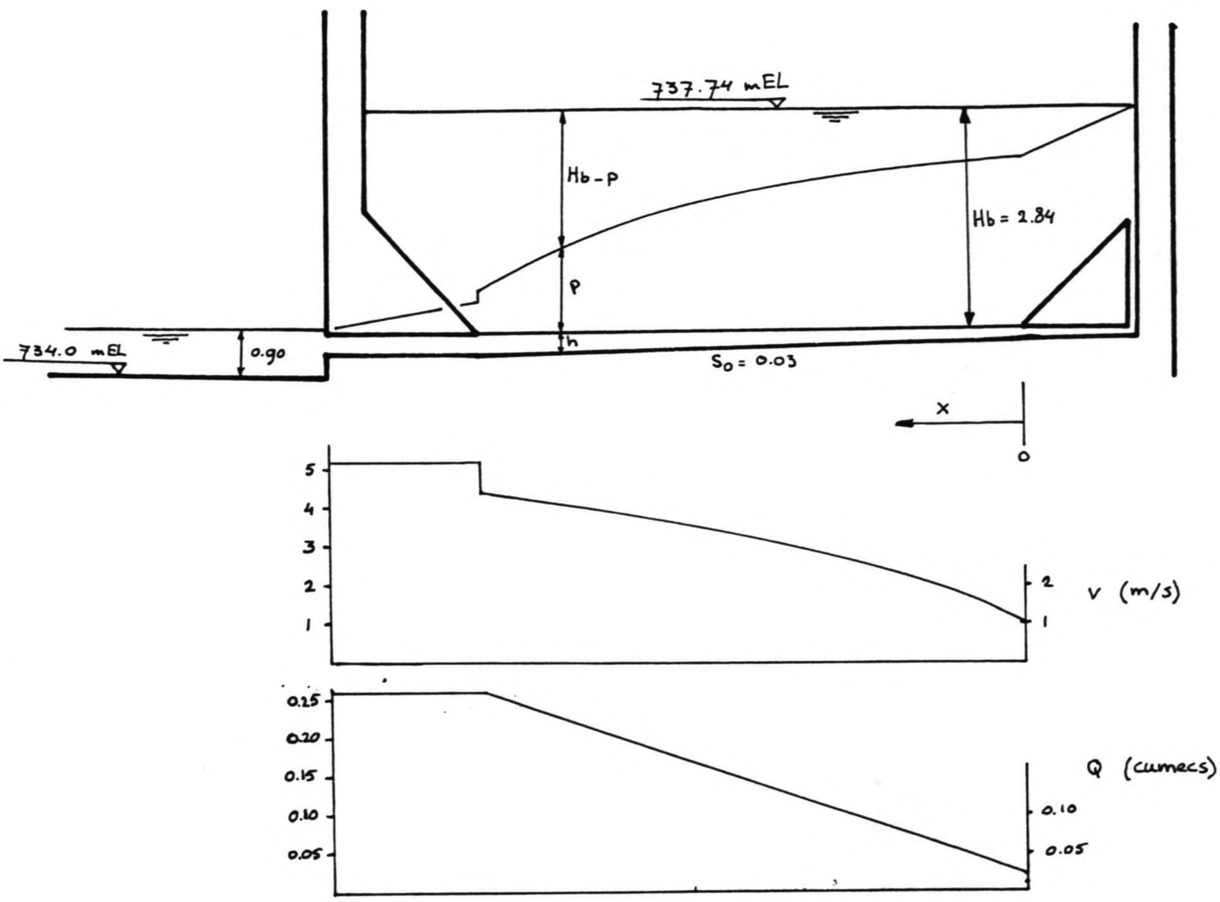


FIGURE 6.10 Flow in flushing channel under minimum flow conditions

The head losses in the pipe leading to the sluiceway are calculated for a pipe diameter **D = 0.25 m.**, and slightly rounded edges at its entrance. For the above case (figure 6.10) they are:

- friction losses: $\Delta H_f = 1 * (v / M*R^{0.67})^2$
  $= 2 * (4.4 / 60*0.156)^2$
  $= 0.44$ m.

- entrance losses: $\Delta H_e = \xi_1 * (v^2/2g)$ (figure 4.18)
  $\approx 0.1\ (14.4^2 / 19.6)$
  $= 0.10$ m.

The initial Q_0 = 0.02 cumecs can be achieved with a pipe of diameter **D = 0.09 m**. The available head at that end of the channel is $(H_b - p)$ = 0.68 m., the pipe length L = 3 m., so the "available" slope is:

$s = (H_b - p) / L = 0.23$

The friction slope of the pipe is:

$$s_f = (v / M * R^{0.67})^2$$

with $M = 95$ (for HDP)
$R = D/4 = 0.023$ m.
$v = Q/A = 3.1$ m/s

so $s_f = 0.18 < 0.23$

6.3. SLUICEWAY

6.3.1. Optimisation criteria and parameters

The **first** and foremost function of the sluiceway is to keep the approach channel deep enough beolw the intake windows. The most important criterion for the dimensioning of the sluiceway therefore is:

- sediment transport capacities in the sluiceway and the approach channel must be high enough to prevent the settling of sediments carried by the river.

Since the sediment transport is dependant on the flow parameter (R * s / Δ * D) [litt.12], this criterion can also be formulated as:

$$(R * s)_{sl} > (R * s)_r \qquad [1]$$

with R = hydraulic radius
s = slope
sl = sluiceway/approach channel
r = river

This criterion must be met during the monsoon season, when the sediment load is considerable. In the dry season it can be neglected.

The **second** function of the sluiceway is to create an area of supercritical flow, to allow the flushing of the desilting basin even under flood conditions. The desired water levels behind the gates have been calculated in section 6.2.2.:

$$WL < 735.0 \text{ m.EL} \qquad (\text{for } Q_r = 30 \text{ cumecs})$$
$$\text{or} \quad WL < 735.8 \text{ m.EL} \qquad (\text{for } Q_r = 500 \text{ cumecs}) \qquad [2]$$

The **third** function of the sluiceway is to transport the ejected sediment from the desilting basin back to the river. This extra sediment load should not lead to exceedance of the transport capacity of the flow in the sluiceway. It is assumed that this is not a problem if:

$$Q_s > 3 * Q_i \qquad [3]$$ (see section 5.5.1)

since the velocities behind the gates will be considerably higher than in the river. Since $Q_f < 0.3\ Q_t = 0.23\ Q_i$ (see section 6.1.2.) this criterion also guarantees that the flushing discharge doesn't disturb the flow pattern in the sluiceway too much, since $Q_f < 0.08\ Q_s$.

The sluiceway will be set lower than the natural river bed level (735.9 m.EL). In order to enable the transport of sediment back into the river the sluiceway bottom level should not be too low:

$$SBL > 734.0 \text{ m.EL} \qquad [4]$$

which is 1 m. below the level of the stilling basin behind the weir.

Also the hydraulic jump should be kept within the length of the sluiceway (≈ desilting basin length = 33 m.), to avoid damage to the river bed behind:

$$x + L_j < 35 \text{ m.} \qquad [5]$$

with x = location and L_j = length of jump.

6.3.2. Discharge formulas

The discharge formula for a radial gate (figure 6.11) is according to Bos [litt.3]:

$$q = c_o * w \sqrt{(2g * H_o)}$$

with q = specific discharge
c_o = discharge coefficient = $\mu / \sqrt{(1 + \mu*w/H)}$
w = gate opening
H_o = backwater depth
μ = contraction coefficient

The water depth in the section of maximum contraction is:

$$h_1 = \mu * w$$

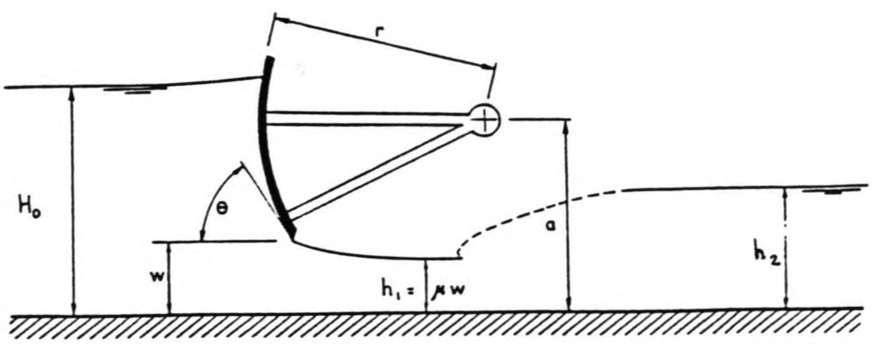


FIGURE 6.11 Radial gate discharge

The contraction coefficient is dependant on the angle of the gate lip θ:

$$\mu = 1 - 0.75\ (\theta/90^\circ) + 0.36\ (\theta/90^\circ)^2$$

with $\theta = \arccos\ \{(a - w)/r\}$
a = gate axis height
r = gate radius

The maximum contraction occurs at a distance of

$$l = w\ /\ \mu$$

The flow behind the gates will be supercritical until the energy losses have resulted in a water depth and velocity that is sufficient for the jump to occur. These losses can be determined with Manning's formula for friction [litt.4]:

$$s_f = (v\ /\ M{*}R^{0.67})^2$$

with s_f = friction slope
$v = Q\ /\ B{*}h$
$M = 60$ (for concrete)
$R = B{*}h\ /\ (B+2h)$

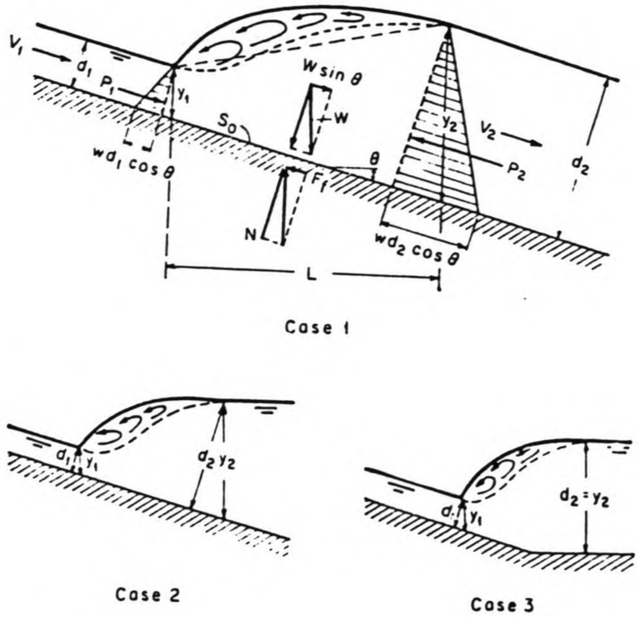
The hydraulic jump occurs when the ratio of the sequent depth h_2 to the initial depth h_1 is a certain value, dependant on the bottom slope and the initial Froude number [litt.4]:

$$y_2\ /\ y_1 = 0.5\ \{\sqrt{(1+8G^2)} - 1\}$$

and $G = F_1\ /\ \sqrt{\{\cos\theta - \tan\theta * K * L_j\ /\ (y_2 - y_1)\}}$

with F_1 = initial Froude number = $v_1\ /\ \sqrt{(g{*}h)}$
θ = bottom slope in degrees
K = correction coefficient for surface profile of the jump
L_j = length of the hydraulic jump
y = vertical component of water depth

These formulas are based on the momentum equation (see figure 6.12).
The values of K and L_j are mainly dependant on the Froude number, and are determined on the basis of experimental data [litt.4]. The resulting graphs that are used for design purposes are presented in Appendix D.



(source: litt.4)

FIGURE 6.12 Parameters of a hydraulic jump

6.3.3. Optimisation of main dimensions

6.3.3.1. Gates and first part of sluiceway

First the size and level of the gates are determined according to criteria [2], [3] and [4], using Q_r = 30 cumecs as the limiting condition, since that is considered to be the minimum river discharge during which the gates are open (see section 6.1.2.).
Secondly these dimensions are checked for the design flood conditions (Q_r = 500 cumecs) to see whether criteria [1] and [2] are met, since the sediment load is at a maximum under those conditions.

When the bottom of the sluiceway is set at 734.30 m.EL in the gate section the water depth before the gate is:

$$H_o = (738.12 - 0.12) - 734.30 = 3.70 \text{ m.}$$

allowing for a head loss of 0.12 m. over the approach channel.
The minimum sluiceway discharge should be according to criterion [3]:

$$Q_s = 3 * Q_i$$
$$= 3 * 9 = 27 \text{ cumecs}$$

Obviously this criterion can not be met when the river discharge itself is only 30 cumecs. The maximum available $Q_s = Q_r - Q_i = 30 - 9 =$ **21 cumecs**. For this extreme case, with low sediment discharge in the river, this should not be a problem.

When the gates are ± halfway lifted, say $w = 1.2$ m., $r = 3.5$ m. and $a = 2.5$ m., the lip angle will be:

$$\theta = \arccos((a-w)/r)$$
$$= \arccos((2.5 - 1.2)/3.5) = 68°$$

and therefore

$$\mu = 1 - 0.75*(68°/90°) + 0.36*(68°/90°)^2 = 0.64$$

According to criterion [2] the depth behind the gates should be:

$$h < (735.0 - 734.3) = 0.70 \text{ m.}$$

This criterion is met when $w = 1.1$ m.:

$$\mu * w = 0.64 * 1.1 = 0.70 \text{ m.}$$

The discharge coefficient then is:

$$c_o = \mu / \sqrt{(1 + \mu*w/H_o)}$$
$$= 0.64 / \sqrt{(1 + 0.70/3.7)}$$
$$= 0.59$$

and the specific discharge:

$$q = c_o * w * \sqrt{(2g * H_o)}$$
$$= 0.59 * 1.1 * \sqrt{(19.6 * 3.7)}$$
$$= 5.5 \text{ cumecs/m.}$$

So the gate width must be at least:

$$B = Q_s / q$$
$$= 21 / 5.5 = 3.8 \text{ m.}$$

Under the design **flood** conditions ($Q_r = 500$ cumecs, BWL = 738.88 m.EL) the gates will be completely open, so $\theta = 90°$, and

$$\mu = 1 - 0.75 + 0.36 = 0.61$$

The water depth behind the gates according to criterion [2] is:

$$h < (735.8 - 734.3) = 1.5 \text{ m.}$$

so $w < 1.5 / 0.61 = 2.46$ m.

This would result in:

$$\Delta H_f \approx 0.18 \text{ m. (estimated)}$$
$$H_o = (738.88 - 0.18) - 734.3 = 4.40 \text{ m.}$$
$$c_o = 0.61 / \sqrt{(1 + 1.5/4.4)} = 0.53$$
$$Q_s = B * c_o * w * \sqrt{(2g * H_o)}$$
$$= 3.8 * 0.53 * 2.46 * \sqrt{(19.6 * 4.4)}$$
$$= 45.7 \text{ cumecs}$$

These dimensions must be checked against criterion [1]. For the flood conditions the river parameters are:

$$Q_r = B * h * v = B * h * C * \sqrt{(h * s)}$$

with $B = 275$ m.
C = Chezy coefficient = 35
$s = 0.006$

so $h = 0.77$ m.

which gives for the flow parameter:

$$\mathbf{(R * s)_r = 0.0046}$$

The sluiceway section in front of the gates has the following parameters:

$$\begin{aligned} B &= 3.8 \text{ m.} \\ v &= Q / B*h = 3.1 \text{ m/s} \\ h &= H_o - v^2/2g \\ &= 4.40 - 0.48 = 3.92 \text{ m.} \end{aligned} \left.\vphantom{\begin{aligned} v \\ h \\ = \end{aligned}}\right\} \quad \text{(by trial and error)}$$

so $R = B * h / (B + 2h) = 0.78$ m.

The equivalent slope can be found with Manning's formula:

$$\begin{aligned} s &= (v / M*R^{0.67})^2 \\ &= (3.1 / (60 * 0.78^{0.67})^2 \\ &= 0.0037 \end{aligned}$$

which gives as flow parameter for the sluiceway:

$$(R*s)_s = 0.0029 \quad < \quad (R*s)_r$$

The dimensions must therefore be chosen differently, to increase the discharge and the velocities in the part of the sluiceway in front of the gates.

The **optimum** is found by trial and error (figure 6.13):

- two gates of 1.8 m. height and 2.1 m. width
- a sluiceway bottom level of 736.0 m.EL, falling just before the gates to the level of 734.0 m.EL.
- a sluiceway width of 5 m.

This results in a maximum Q_s = 39 cumecs and a velocity v = 4.5 m/s under the flood conditions. The flow parameter then is:

$$\begin{aligned} h &= H_o - v^2/2g = 1.75 \text{ m.} \\ R &= 1.03 \text{ m.} \\ s &= 0.0054 \end{aligned}$$

so $(R*s)_s = 0.0055 > (R*s)_r = 0.0046$

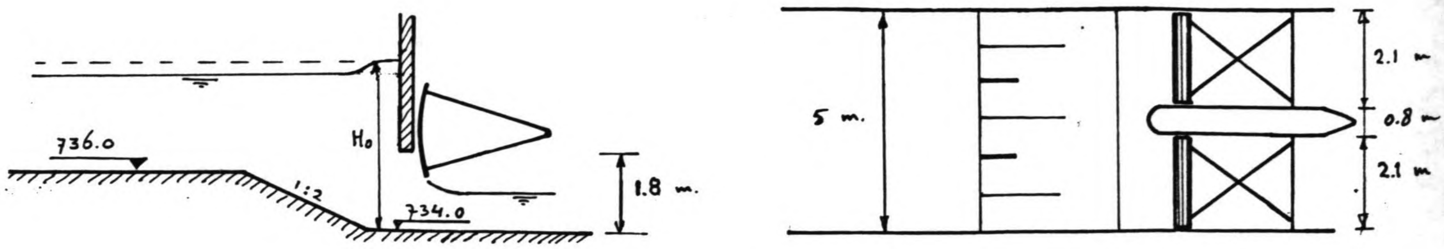


FIGURE 6.13 Dimensions of sluiceway gates

The extra losses due to the expansion are:

$$\Delta H_e = \xi_i (\Delta v)^2 \ / \ 2g \qquad \text{(see figure 6.5)}$$

and $\Delta v = v_1 * (1 - h_1/h_2)$

so $\Delta H_e = 0.68 * \{4.5*(1 - 1.75/3.75)\}^2 \ / \ 2\,g$
$= 0.20$ m.

and the friction losses:

$$\Delta H_f = s * l = 0.0046 * 40 = 0.18 \text{ m.}$$

so the depth in front of the gates is:

$$H_o = 4.88 - 0.20 - 0.18 = 4.50 \text{ m.}$$

The width of sluiceway and gates imply a dividing wall of (5.0 - 2 * 2.1) = 0.8 m., strong enough to support the gates (see Appendix E).

Checking criterion [1] under the minimum flow conditions (Q_r = 30 cumecs) gives the necessary gate opening:

Q_s = 21 cumecs
H_o = 4.00 m. }
h = 1.75 m. } (by trial and error)
v = 2.4 m/s }
s = 0.0015

so $(R*s)_s = 0.0015 \quad > \quad (R*s)_r = 0.0007$

if the gate opening is: w = 1.0 m.

A wider gate opening would lower the water level in the sluiceway in front of the intake windows. This would reduce their efficiency.

With the operation of the gates as proposed below the

water levels in the intake window section could be kept constant, which is a convenient operating procedure:

TABLE 6.1 SLUICEWAY GATE OPERATION AND FLOW PARAMETERS							
Q_r [cumecs]	Backwater level [m.EL]	H_o [m]	Q_s [cumecs]	Gate opening [m]	Water level [m.EL]	Velocity [m/s]	Velocity head [m]
30	738.12	4.00	21	1.0	737.75	2.4	0.29
100	738.30	4.10	27	1.25	737.75	3.1	0.50
200	738.46	4.20	29	1.4	737.75	3.4	0.57
300	738.62	4.30	34	1.6	737.75	3.9	0.77
400	738.76	4.40	36	1.7	737.75	4.2	0.89
500	738.88	4.50	39	1.8	737.75	4.5	1.02

intake window section

6.3.3.2. Sluiceway behind the gates

The dimensions of this part of the sluiceway should be optimised within the limitations of criteria [1], [2], [4] and [5]. Criterion [1] will be met if the hydraulic jump occurs in the second half of the sluiceway, since the flushing pipe outlets are located in the half (figure 6.9).

A spreadsheet program was used to compute the sluiceway flow, incorporating the formula's presented in section 6.3.2. (see Appendix D).
It appeared that the limiting factor is the fixation of the hydraulic jump. The tailwater level is insufficient to create the jump under flood conditions (table D.2).

Usually in such a case some kind of energy dissipation is created by sills, chute blocks, sudden drops etc.. In this case this approach has not been chosen, because it would hamper the transport of cobbles and stones. Floor blocks would also wear down quickly in the fast-flowing soiled water. The floor of the sluiceway should therefore be relatively smooth, without sudden changes in the vertical alignment.

For these reasons it is considered wise to fix the location of the jump by creating an oblique jump. A sudden horizontal contraction creates a crosswave when the supercritical flow hits the vertical face of the wall (see figure 6.16).

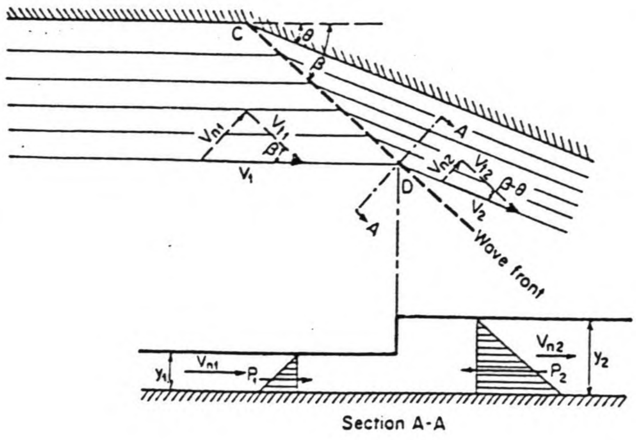


FIGURE 6.14

Oblique jump

(source:litt.4)

If the shock front is high enough a hydraulic jump will occur. This is dependent on the angle Θ, the initial depth h_1 (or y_1) and the initial Froude number F_1. The four-quadrant graph shown in figure 6.15 gives the necessary relationships (source: litt.4).

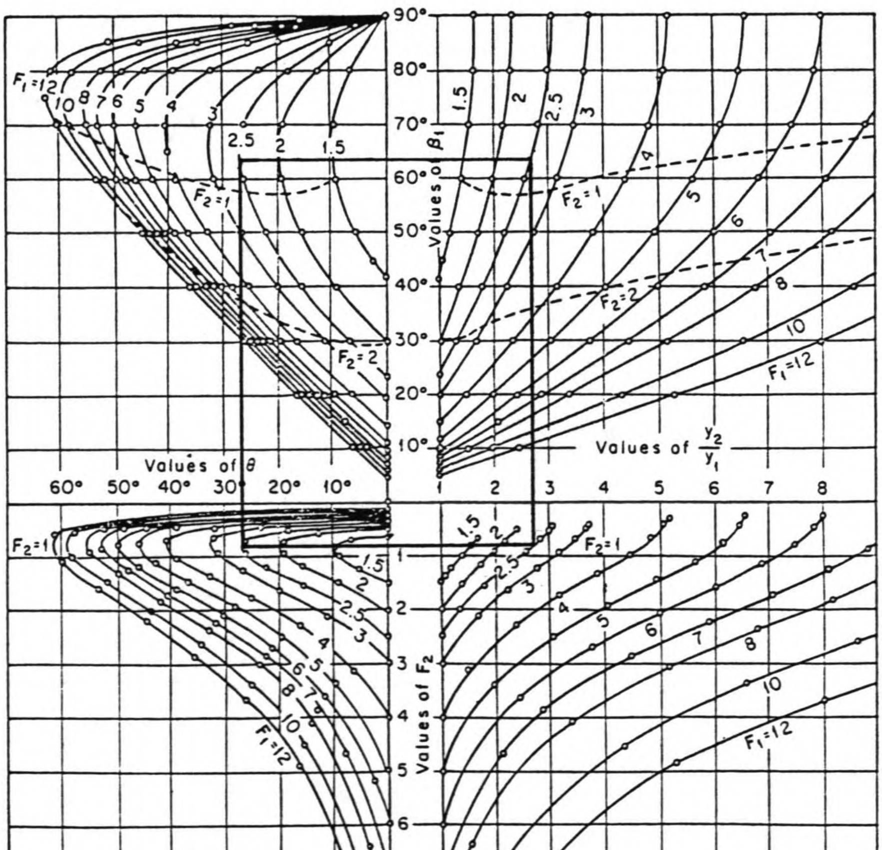


FIGURE 6.15 Relationships between crosswave parameters

If the crosswave is not high enough for the jump it reflects off the opposite wall and travels downstream, until the jump does occur (figure 6.16).
Negative disurbances are created by the second wall corner (point D). To avoid these complications the dimensions are chosen here to create the jump at the first shockfront.

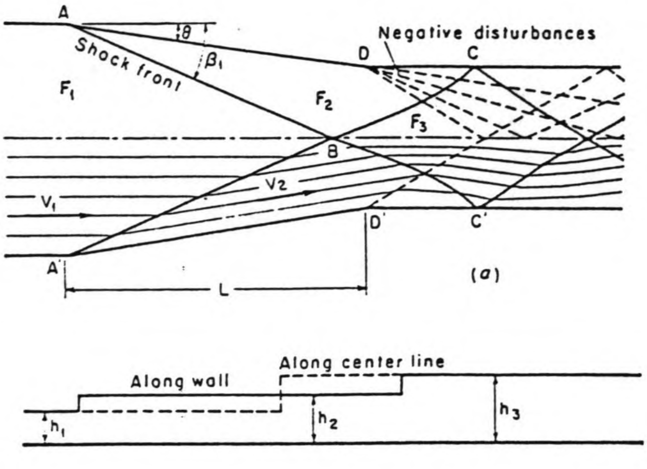


FIGURE 6.16 Crosswaves in a contraction (source: litt.4)

(For the following computations the section numbers of figure 6.19 are used.)

The tailwater level under the design flood conditions is 736.67 m.EL (table 2.3), so the sequent depth can be estimated to be:

$$h_3 = (736.67 - 734.00) + \Delta H_f$$
$$= 2.70 \quad \text{m.}$$

assuming $\Delta H_f \approx 0.03$ m.

The initial depth h_2 must be computed from the upstream end, since the flow is supercritical. An important factor is the expansion just behind the gates, due to the end of the dividing wall (figure 6.17).

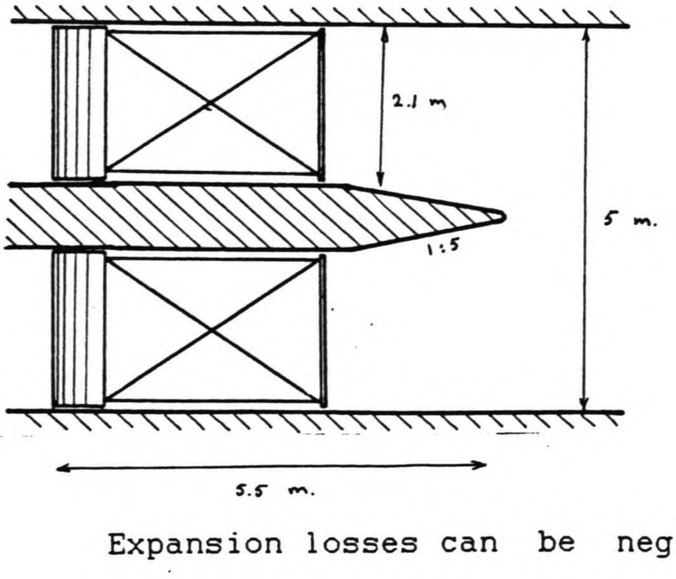


FIGURE 6.17 Shape of dividing wall end

Expansion losses can be neglected if seperation of flowlines from the walls is avoided. According to Chow [litt.4] this is the case if the angle is chosen ± 1 : 5, and the edges are rounded off.

The flow in this part can thus be computed using the Bernouilli equation, the continuity principle, and Manning's formula for friction [litt.4].

At $x = w/\mu = 3.0$ m.: (see figure 6.19)

	$Q = Q_s / 2$	=	19.5	cumecs
	$h_o = \mu * w$	=	1.10	m.
	$b_o =$	=	2.1	m.
so	$v_o = Q/(b*h)$	=	8.4	m/s
and	$H_o = h + v^2/2g$	=	4.70	m.
and	$s_f = (v/M*R^{0.67})^2$	=	0.045	

At x = 5.5 m. this leads to:

	$Q = Q_s$	=	39	cumecs
	$b_1 =$	=	5.0	m.
	$H_1 = H_o - \Delta H_f$	=	4.59	m.
so	$h_1 =$	=	0.92	m. (by trial and error)
and	$v_1 =$	=	8.5	m/s
so	$s_f =$	=	0.034	

The contraction is located at x = 17.5 m., where

	$H_2 = H_1 - \Delta H_f$	=	4.18	m.
so	$h_2 =$	=	0.99	m.
and	$v_2 =$	=	7.9	m/s
so	$F_2 = v / \sqrt{(g*h)}$	=	2.5	

We now know the initial depth h_2 and the sequent depth h_3 of the oblique jump (= y_1 and y_2 in figure 6.15), and the initial Froude number F_2 (= F_1 in the figure).

According to figure 6.15 the combination of

$h_3/h_2 = 2.70/0.99 = 2.7$
and $F_2 = 2.5$

leads to:

$\theta = 27°$
$\beta = 63°$
and $F_3 = 0.8$
so $v_3 = F * \sqrt{(g*h)} = 4.1$ m/s
and $b_3 = Q / (v*h) = 3.50$ m.

These results are shown in figure 6.18.

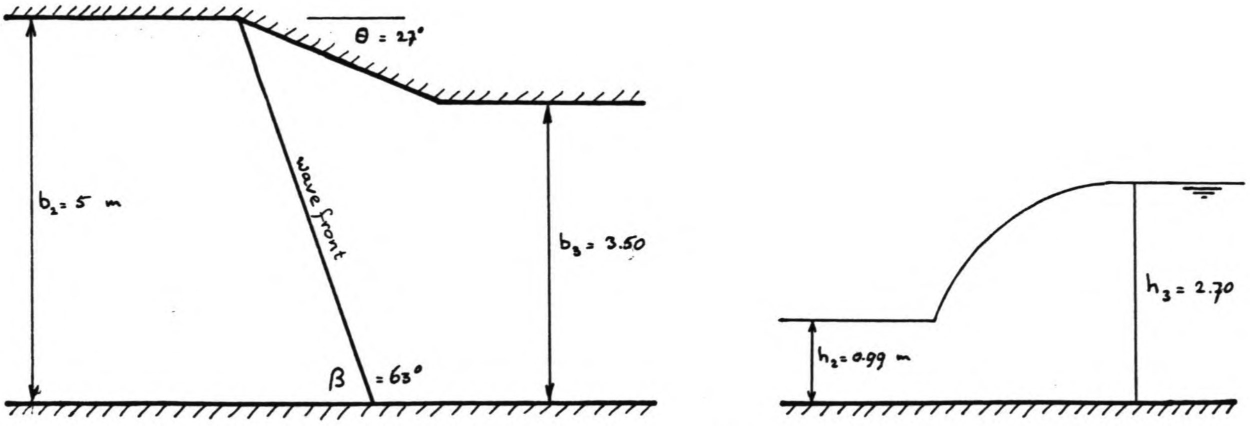


FIGURE 6.18 Oblique jump in sluiceway

After the contraction the sluiceway floor slopes upward, to reach the level of the launching apron behind the weir (= 734.80 m.EL). To avoid reaching the critical velocity again this rising of the floor must be combined with increasing the width of the sluiceway (figure 6.19). The critical velocity for h_4 = (736.67 - 734.80) = 1.87 m. at the outlet section is:

$v_c = \sqrt{(g*h)} = 4.3$ m/s
so $b_c = Q/(v_c*h) = 4.85$ m.

The outlet width is therefore set at:

$b_4 = 5.2$ m.
so $v_4 = 4.0$ m/s
and $F_4 = 0.94$

This is cutting it close, but a greater width and lower velocity would reduce the sediment transport capacity too much (see below).

Between sections (3) and (4) the parameters are:

$v_{avg} = (4.1 + 4.0)/2 = 4.1$ m/s
$R_{avg} = (1.06 + 1.07)/2 = 1.07$ m.

so $s_f = (v/(M*R^{0.67}))^2 = 0.004$
and $\Delta H_f = s_f * 1 = 0.04$ m.

so the initially assumed ΔH_f = 0.03 m. is close enough.

The resulting water levels under the design flood conditions are shown in figure 6.19.

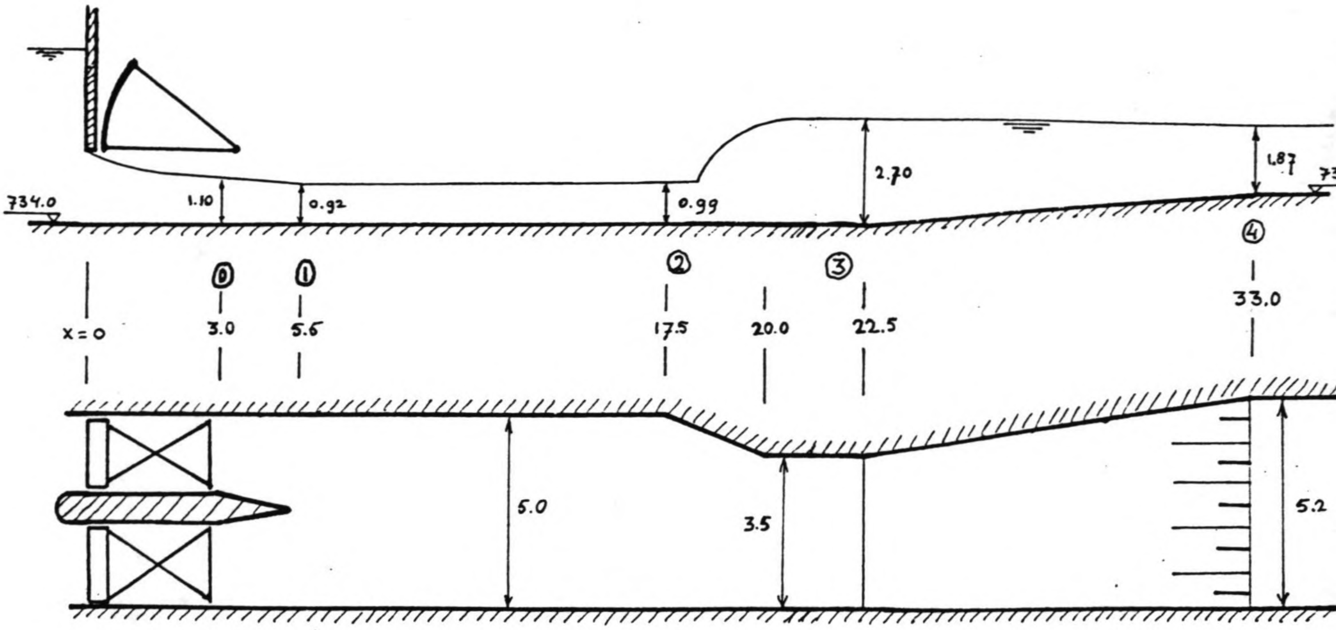


FIGURE 6.19 Flow profile in sluiceway

These dimensions must now be checked against criterion [1], the sediment transport capacity. Obviously this is only necessary for the subcritical flow.
In section (3) the parameters are:

v_3 = 4.1 m/s
R_3 = 1.06 m.
so s_f = 0.0043
and $(R*s)_s$ = 0.0046 = $(R*s)_r$ = 0.0046

In section (4) the parameters are:

v_4 = 4.0 m/s
R_4 = 1.07 m.
so s_f = 0.0041
and $(R*s)_s$ = 0.0043 < $(R*s)_r$

so the transport capacity could be slightly sufficient. The flow is very turbulent however, so it will most probably not be a problem, since these conditions only occur for short periods.

Finally the dimensions are checked for Q_s = 21 cumecs and minimum river water discharge during the period monsoon: Q_r = 30 cumecs. Normally the tailwater level would be TWL = 736.00 m.EL (see table 2.3). Since there is no water flowing over the weir in this situation however another water level will occur. Because of the complicated (three-dimensional) flow behind the sluiceway the water levels cannot easily be computed. An educated guess must be made, which should be checked in model studies.

The specific discharge shortly after the sluiceway outlet (at the end of the launching apron, where the bottom rises to the river bed level) is about 7 times higher than when the full discharge would flow over the weir. On this basis the water level that matches Q_r = 200 cumecs is assumed:

h_4 = 736.30 m.EL (see table 2.3)

Using the same methods as presented above the computation gives the results shown in table 6.2.

TABLE 6.2 FLOW PARAMETERS UNDER LOW FLOW CONDITIONS

x	b	h	v	R	s	ΔH	H	F	(R*s)
1.6	2*2.1	0.64	7.8	0.40	0.058		3.75	3.1	0.023
						↓ 0.23			
5.5	5.0	0.55	7.6	0.45	0.047		3.52	3.3	0.021
						↓ 0.56			
17.5	5.0	0.62	6.8	0.47	0.035		2.96	2.8	0.016
20	3.5	2.38	2.5	1.01	0.002		2.70	0.5	0.002
						↑ 0.03			
33	5.2	1.50	2.7	0.95	0.002		2.67	0.7	0.002

Since the flow parameter of the river under these conditions is :

$(R*s)_r$ = 0.10 * 0.006 = 0.0006

there is obviously no danger of sediment deposition.
The velocity at the outlet also stays well below the critical value. This is of course dependent on the validity of the assumed water depth. A 0.20 m. lower tailwater level would still give sub-critical flow however (F = 0.9).

6.4. FOUNDATION AND BED PROTECTION

6.4.1. Seepage control

A weir founded on pervious soils will inescapably suffer from underseepage, unless a cut off wall or grouted screen under the weir can reach an impervious layer. In the case of the Jhimruk Khola the bedrock is about 17 m. below the surface, so for all practical purposes it can be considered out of reach. The underseepage can be reduced considerably however with the use of cut off walls and/or clay blankets.

The NEA design provides for two cut off walls, one on the upstream end of the weir and one on the downstream end of the stilling basin. They can be replaced by a clay blanket in front of the weir (figure 6.20). This is a more cost effective solution, easier to construct and using local materials. It must be looked into whether sufficient supply of clay is available in the immediate vicinity of the project.

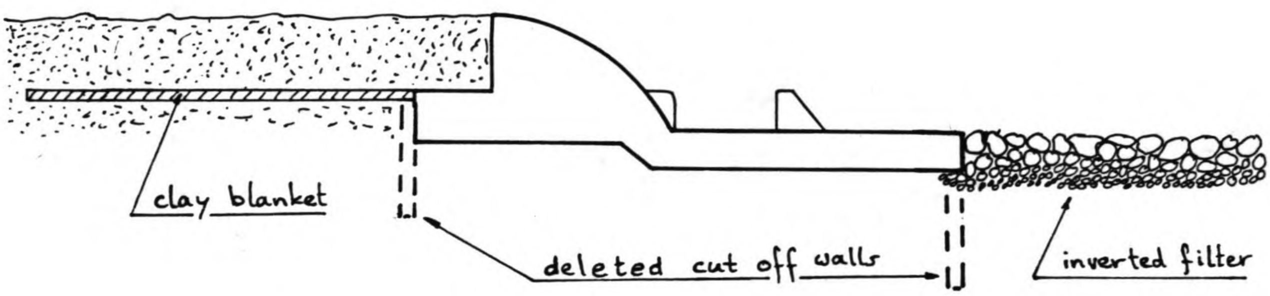


FIGURE 6.20 Clay blanket and inverted filter

The blanket will be covered with deposits after the construction of the weir, so no protection against scour is necessary. It is estimated that a 10 m. long 40 cm. thick blanket has the same effect as 2.5 m. deep cut off walls, in combination with the longer stilling basin (see below).

Drawing 2 shows that the blanket should be connected to the concrete floor of the sluiceway to avoid seeping around that end of the weir. Clay blanket and sluiceway floor are constructed on the same level (736.0 m.EL).

To protect the toe of the structure against piping an inverted filter should be applied there, in combination with a gabion launching apron.

6.4.2. Stilling basin

As stated in section 3.3.3. it is advised not to use floor blocks for the stilling basin behind the weir. The level and length of the stilling basin are determined with the 50-year flood as limiting condition: (see tables 2.1 and 2.3):

Q_r = 1500 cumecs
BWL = 739.94 m.EL
TWL = 737.62 m.EL

The water depth h_1 (figure 6.21) can be found by trial and error:

H_1 = 739.94 − 735.0 − ΔH
$\Delta H \approx 0.1 * v^2/2g$ = 0.39 m. [litt.11]
so H_1 = 4.55 m.
h_1 = 0.69 m.
v_1 = 8.7 m/s ($v^2/2g$ = 3.86 m.)
F_1 = 3.3

The tailwater depth h_u = 737.62 − 735.0 = 2.62 m.

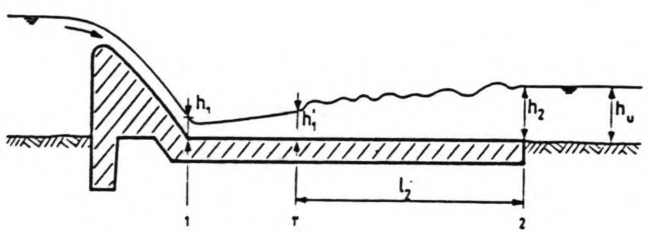


FIGURE 6.21

Hydraulic jump behind weir

(source: litt.11)

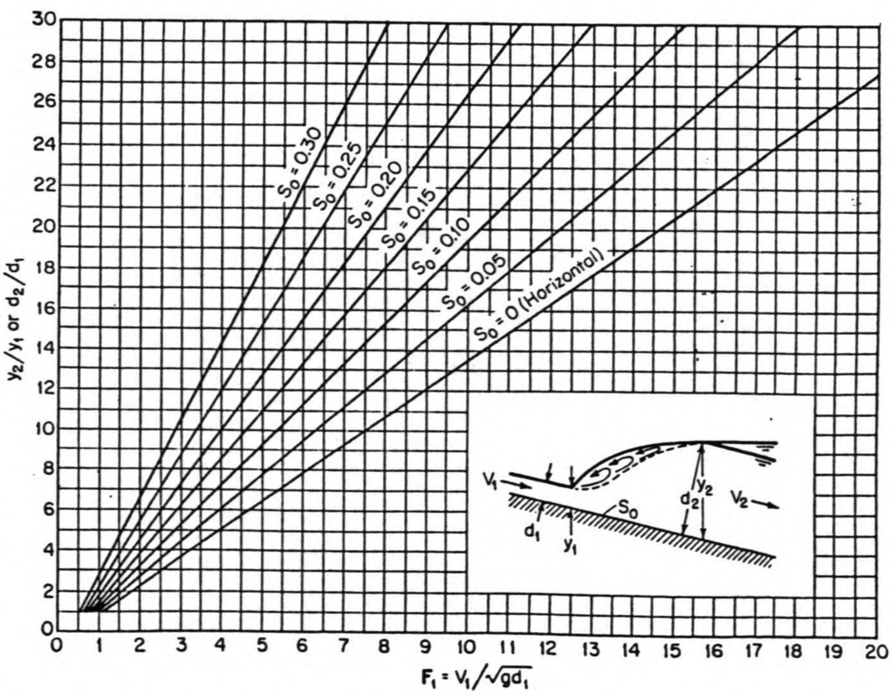


FIGURE 6.22 Sequent depths (source: litt.4)

With the use of figure 6.22 it can be checked if the hydraulic jump will occur at the toe of the weir:

$h_2/h_1 \approx 4$

so $h_2 = 2.76$ m. $>$ $h_u = 2.62$ m.

The conclusion is that the level of the stilling basin must be **lowered to 734.8 m.EL**. In that case the parameters are:

$h_1 = 0.67$ m.
$v_1 = 8.9$ m/s
$F_1 = 3.4$
$h_2 = h_u = 2.82$ m.

The necessary length of the stilling basin can be found with figure 6.23, which gives the length of the hydraulic jump:

$l_2/h_2 \approx 5.5$

so $l_2 = 15.5$ m.

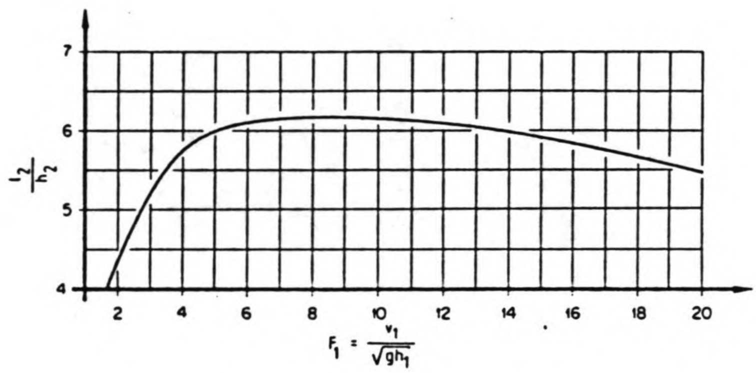


FIGURE 6.23 Length of hydraulic jump (source litt.11)

It is not absolutely necessary to contain the whole jump within the stilling basin however [litt.5]. A more economic size is assumed to be:

$l_2 = 13$ m.

which will contain more than 80 % of the jump. The remaining energy should not be harmful to the gabion launching apron behind the stilling basin.

6.4.3. Stability

The stability of the desilting basin and the sluiceway must be checked against **sliding** and **lifting**. Basin and sluiceway are for that purpose considered to be one structure. The figures 6.24.a to 6.24.c show several combinations of loads:

- case a. Design flood for structural integrity:
  BWL = 740.6 m.EL
  TWL = 738.2 m.EL
  Plant out of operation, so basin WL = BWL
  Sluiceway gates closed, so sluiceway WL =BWL
  The section is taken just before the gates.

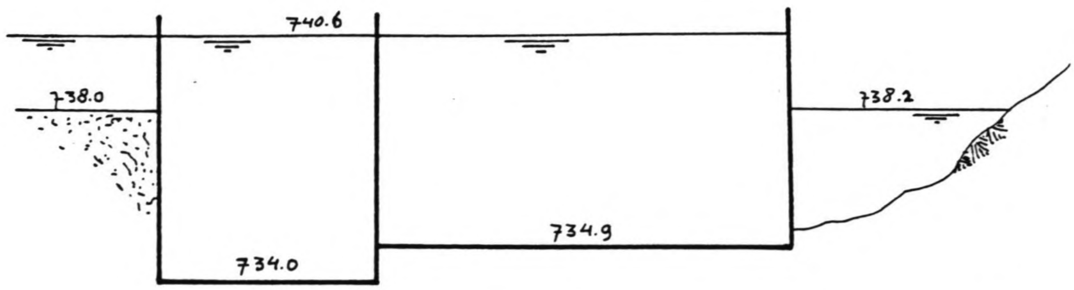


FIGURE 6.24.a

- case b. The same flood conditions as a.
  Basin is empty because of maintenance activities.
  Sluiceway gates are open.
  The section is taken behind the gates.

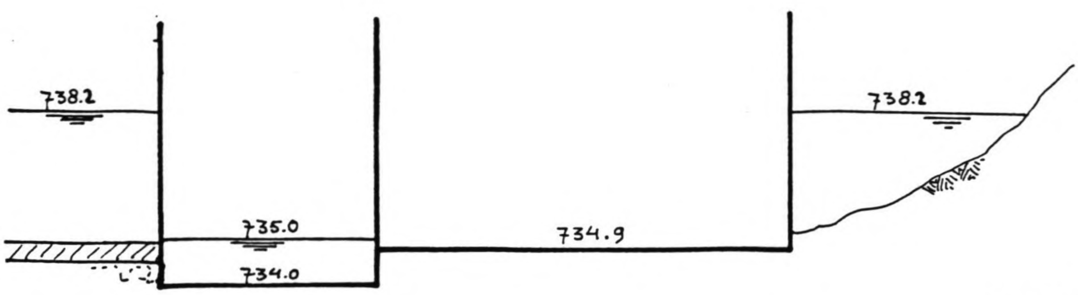


FIGURE 6.24.b

- case c. Maximum river discharge under which the system is operated: TWL = 736.7 m.EL
  Basin WL = 737.9 m.EL (see section 6.2.1.4)
  Sluiceway water depth in supercritical flow ≈ 1.0 m. (see section 6.3.3.2).

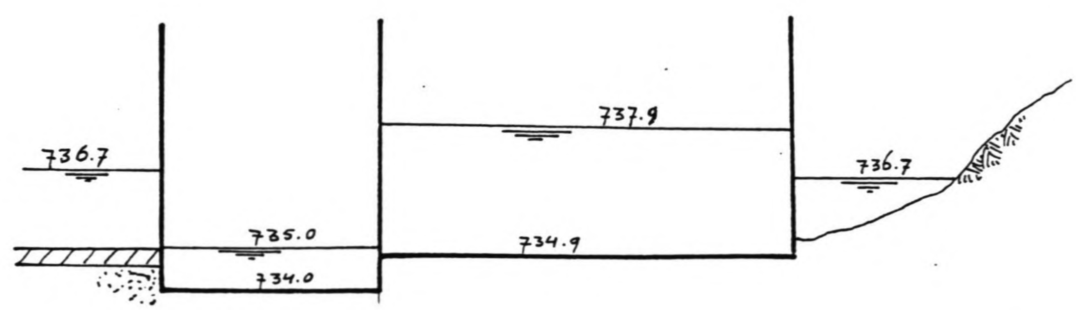


FIGURE 6.24.c

The area between the desilting basin and the hillside is connected to the downstream river bed by a culvert underneath the first part of the tunnel, to allow the drainage of run off water from the hill (see drawing 2). The water level on that side of the basin is therefore identical to the tailwater level.

6.4.3.1. Lifting

For **lifting** the situation with maximum uplift is the most disadvantageous (case b.). This is a very rare case because of the improbable combination of events. For this reason it is not considered necessary to introduce a safety factor. Conservative assumptions have been made however on dimensions, specific weight, hydrostatic uplift etc., because of the less than perfect building practices.

The basin structure is divided into three segments by expansion joints. Each segment should be stable on its own. The stability against lifting is checked for a cross section of the middle segment (where case b. is possible). The hydrostatic pressures acting on this section are shown in figure 6.25.

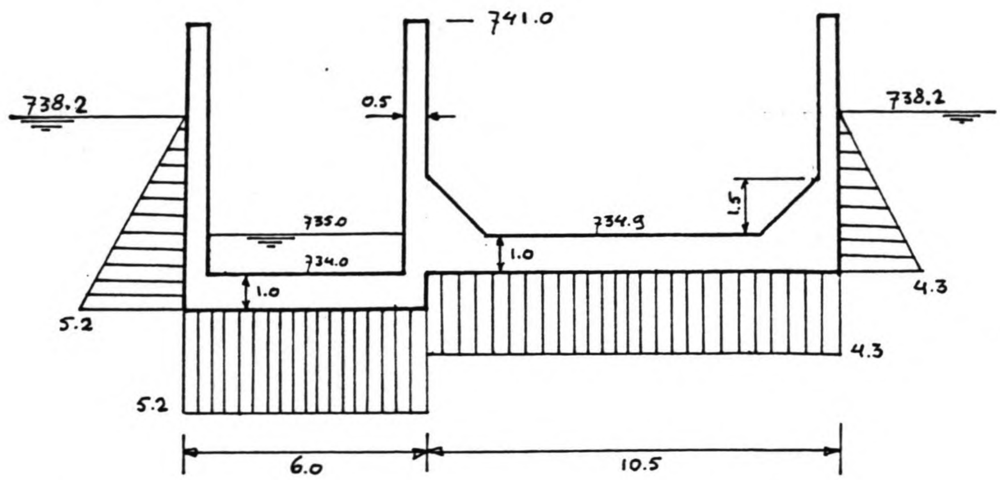


FIGURE 6.25. Hydrostatic pressures

Per meter basin length the total lift is:

$$L = \Sigma(p * B)$$
$$= (6*5.2 + 10.5*4.3) * 9.8$$
$$= 748 \text{ kN}$$

With 500 mm thickness the weight of the walls is:

$$W_w = \Sigma(b * h) * \rho_c * g$$
$$= \{2*(0.5*7.0) + 0.5*6.1 + 2*(1.5^2/2)\} * 2.4 * 9.8$$
$$= 289 \text{ kN}$$

There is some extra weight of gates. ladders, platforms, reinforcement steel etc.:

$$W_e = 15 \text{ kN}$$

The weight of the water in the sluiceway is:

$$\begin{aligned} W_{wat} &= B_s * h * g \\ &= 5.0 * 1.0 * 9.8 \\ &= 49 \text{ kN} \end{aligned}$$

To prevent lifting the total weight must be at least equal to the total lift:

$$\Sigma W > L$$

so the floor weight must be at least:

$$\begin{aligned} W_f &= L - W_w - W_e - W_{wat} \\ &= 748 - 289 - 15 - 49 \\ &= 395 \text{ kN} \end{aligned}$$

and therefore the floor thickness must be:

$$\begin{aligned} h &= W_f / (B * \rho_c * g) \\ &= 395 / (16.5 * 2.4 * 9.8) \\ &= \mathbf{1.0} \text{ m.} \end{aligned}$$

6.4.3.2. Sliding

The first segment (intake chamber + transition) must be checked for **sliding**, since only in that part there is a difference in water level between both sides of the structure (figure 6.26). It is assumed that the hydrostatic pressure on the floor is distributed linearly between both extremes, taking into account the pressure reduction by the clay blanket.

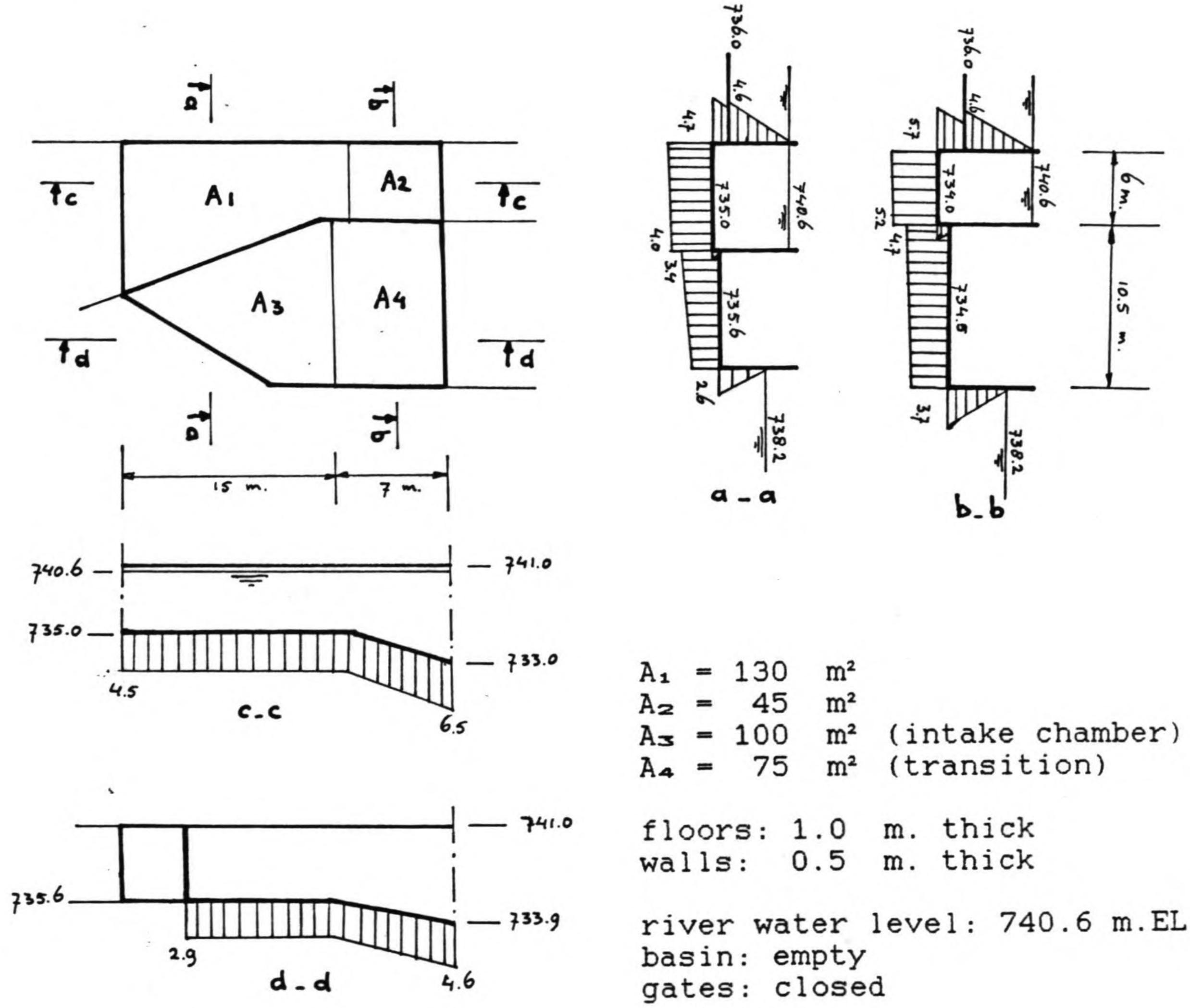


FIGURE 6.26. Limiting case for sliding

The resulting horizontal force is:

$$
\begin{aligned}
F &= \Sigma(b * p * h) \\
&= \{22*4.6^2/2 + 15*4.2*1 + 7*4.7*2 \\
&\quad - 15*2.6^2/2 - 7*3.7^2/2 \\
&\quad - 15*3.1*0.6 - 7*4.5*0.5\} * 9.8 \\
&= 2150 \text{ kN}
\end{aligned}
$$

The weight of the walls is:

$$W_w = \Sigma(h * l) * b * \rho_c * g$$
$$= \{15*5.0 + 7*6.0 + (16+17)*4.4 + 2*7*5.3\} * 0.5 * 2.4 * 9.8$$
$$= 3950 \text{ kN}$$

The weight of the floor is:

$$W_f = \Sigma A * h * \rho_c * g$$
$$= \{130 + 45 + 100 + 75\} * 1.0 * 2.4 * 9.8$$
$$= 8230 \text{ kN}$$

The extra weight for miscellaneous structures is deleted against the reduction of weight for the intake windows.

The weight of the water in the sluiceway is:

$$W_{wat} = \Sigma(A * h) * g$$
$$= \{130 * 4.6 + 45 * 5.6\} * 9.8$$
$$= 8330 \text{ kN}$$

The total lift is:

$$L = \Sigma(A * \Delta H) * g$$
$$= \{130*4.4 + 45*5.4 + 100*3.0 + 75*4.2\} * 9.8$$
$$= 14000 \text{ kN}$$

$$\text{So } (W-L) = W_w + W_f + W_{wat} - L$$
$$= 3950 + 8230 + 8330 - 14000$$
$$= 6510 \text{ kN}$$

The stability limit for sliding is:

$$F / (W - L) < \tan \emptyset$$

with $\emptyset$ = angle of internal friction (= 30° for sand).

In this case:

$$F / (W - L) = 2150 / 6510$$
$$= 0.33 \quad < \tan 30° = 0.58$$

6.4.4. Foundation strength

The pressure on the foundation soil should not exceed the bearing capacity, which is (see section 2.3):

50 kN/m at the right bank
500 kN/m at 15 m. from the right bank.

The maximum ground pressure occurs when the basin is full, and in a section behind the weir (figure 6.27).

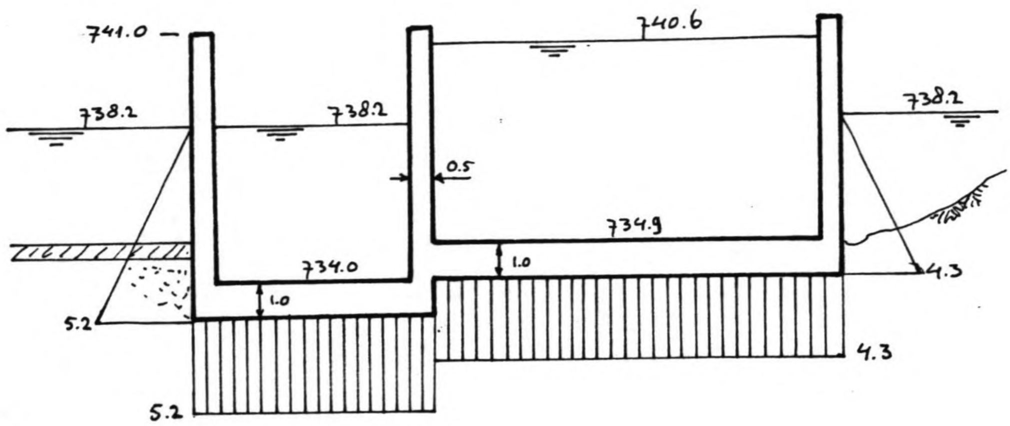


FIGURE 6.27. Limiting case for foundation strength

In this case the total lift is:

$$L = \Sigma(p * B)$$
$$= \{5.2*6 + 4.3*10.5\} * 9.8$$
$$= 748 \text{ kN}$$

The weight of the concrete structure is:

$$W_c = \Sigma(h * b) * \rho_c * g$$
$$= \{7.0*0.5 + 6*1.0 + 2*(6.1*0.5) + 10.5*1.0\}*2.4*9.8$$
$$= 614 \text{ kN}$$

The weight of the water in basin and sluiceway is:

$$W_{wat} = \Sigma(h * B) * g$$
$$= \{4.2*5 + 5.7*10\} * 9.8$$
$$= 764 \text{ kN}$$

So $(W - L) = W_c + W_{wat} - L$
$$= 614 + 764 - 748$$
$$= 630 \text{ kN}$$

and $\sigma = (W - L) / B$
$$= 630 / 16.5$$
$$= 38 \text{ kN/m} \qquad < 50 \text{ kN/m}$$

Local pressures can be higher than this average, but there is an ample safety margin.

7. COST COMPARISON

The costs of the proposed alterations in the design should not increase the total costs of the project, since the Benefit Cost Ratio is already low (see section 2.4). A rough cost estimate is given here for the alternative design of the intake and desilting structures. The unit rates given in the NEA report are used, because **comparison** of the designs is the object of this exercise. In actual fact considerable savings can be achieved by employing more local workforce and different construction techniques.

The bill of quantities is roughly estimated on the basis of the main dimensions.
For the **reinforcement steel** an average percentage of 1 % is assumed, in accordance with the assumptions in the NEA figures. Comparison with the reinforcement percentages calculated in Appendix E shows that 1 % is clearly on the safe side.
The **overburden excavation** includes the approach channel leading from the (future) winter bed to the intake. This will have to be constructed after the rising of the river bed upstream of the weir.

TABLE 7.1. CONSTRUCTION COSTS			(US $, 1986 prices)	
	Quantity		Unit rate	Costs
Overburden excavation	5000	(m^3)	5	25,000
Backfill	500	(m^3)	7	3,500
Stones (Lanching apron and gabions)	1500	(m^3)	9.5	14,000
Wire mesh (gabions)	3000	(m^2)	10	30,000
Graded filter materials	250	(m^3)	11.5	3,000
Structural concrete	1400	(m^3)	130	182,000
Reinforcement steel	110,000	(kg)	1.1	121,000
Formwork	2000	(m^2)	18	36,000
Steel - trashrack	350	(kg)	3	1,000
- gates	3100	(kg)	5	15,500
Stoplogs	2	(m^3)	590	1,000
TOTAL				**432,000**

Added to this figure must be the costs of the extra 20 m. length of the weir. The NEA report gives as total costs of the 230 m. long weir: US $ 942,000, so the price per meter length is US $ 4100. In the alternative design the weir will therefore cost 20 * 4100 = **US $ 82,000** more.

On the other hand the NEA designed intake and desilting basin can be deleted. The forebay between the desilting basin and the penstock must be replaced by a surge tank or shaft. It is assumed that the costs of forebay and surgetank would be similar (about US $ 150,000).

Table 7.2 summarises the changes.
It shows that the alternative design actually costs less than the NEA design.

TABLE 7.2. COSTS OF ALTERNATIVES	(US $, 1986 prices)	
	<u>NEA</u>	<u>Alternative</u>
Weir	942,000	1,024,000
Intake	429,000	432,000
Desilting basin	207,000	
TOTAL	**1,578,000**	**1,456,000**

8. CONCLUSIONS AND RECOMMENDATIONS

The design of the Jhimruk Hydro-electric Project, as presented by the Nepal Electricity Authority in its Feasibility Study Final Report (1987), has been studied regarding its civil engineering aspects.
The main **conclusions** are:

- The desilting basin should be relocated to the right bank of the Jhimruk Khola.

- The design of masonry weir and bed protection can be simplified, with the use of a clay blanket in stead of cut off walls. The stilling basin should be lengthened and its floor blocks deleted.

- The wooden flashboards should be higher, to enable the reclaiming of cultivated land in the pondage area.

- The Tyrolian bottom intake is unsuited for this project. It will be clogged with sediment during the monsoon period. This can only be remedied by raising the weir crest with ± 2.5 m., which would make the project uneconomical.

- The tunnel can have a smaller cross section and slope, and should be constructed with masonry lining.

- The turbines can be installed in an underground powerhouse 15 m. below the proposed level.

An **alternative design** has been made for the intake and desilting arrangement, consisting of:

- an approach channel with artificial bend,

- a lateral intake in the outer bank of the bend, with intake windows raised above the approach channel floor,

- a sluiceway with radial gates for flushing the area in front of the intake windows,

- a desilting basin directly behind the intake, with continuous flushing during the monsoon period,

- a deepened sluiceway adjacent to the desilting basin, with a sudden contraction to fix the location of the hydraulic jump, and sloping upward to the river bed level,

- a flushing system with lateral channels in the floor of the desilting basin, coming out shortly behind the sluiceway gates.

The dimensions of the design are based on the limiting river discharges and levels:

- minimum level in dry season = weir crest level,
- minimum discharge during monsoon is defined as the 90 % reliable flow in july (driest monsoon month),
- maximum flood during which turbine operation should be possible is the 1 : 5 year flood,
- maximum flood for structural safety is the 1 : 1000 year flood.

The design is based on simple hydraulic theories, and is roughly checked for sediment transport problems.
The **conclusions** are:

- The alternative approach is technically feasible within the limits set above,
- The necessary construction techniques are not complicated, and within the experience of Nepali contractors,
- The operation procedure of the sluiceway gates is simple and straightforward, and the system is not too sensitive to misoperation,
- The sediment transport in the sluiceway will be sufficient for all river conditions under which the system is operated,
- The cutting in the hill slopes is minimal, so the their stability is not endangered,
- The estimated construction costs are slightly lower than those of the original NEA design.

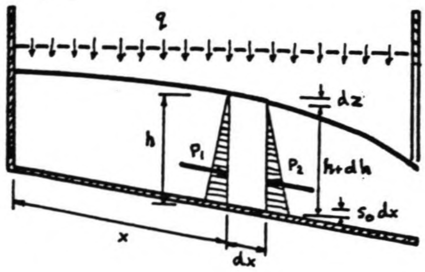
It is **recommended** that BPC incorporates the alternative design presented here in its design and financing activities. Before any commitments are made it should be checked in model studies however.
The attention should be focussed on:

- the amount and composition of the sediment discharge in the river,
- the ability to keep the approach channel between the upstream winter bed and the intake open with the proposed gate size and operation,
- the sediment transport capacity of the sluiceway, especially regarding the level of the deepened part,
- the efficiency of the flushing channels,

since these are the most uncertain aspects of the design.

APPENDIX A. COMPUTATION OF FLOW IN TYROLEAN BOTTOM INTAKE

The Tyrolean bottom intake has a trench built into a weir, with an outlet on one side and with an evenly distributed discharge entering through a grid on top of it. This is a case of spatially varied flow with increasing discharge. For the computation of flow in such cases the momentum equation must be used, as for the sections shown in figure A.1.



q = specific discharge
h = water depth
z = water surface level
s_o = bottom slope
p = hydrostatic pressure

FIGURE A.1. Spatially varied flow

According to Ven Te Chow [litt.4] the dynamic equation for spatially varied flow with increasing discharge is:

$$dh/dx = \frac{s_o - 2\, q^2 * x / (g*A^2)}{1 - q^2 * x^2 / (g*A^2*R)} \qquad [1]$$

with A = h * b
R = hydr. radius
= A /(b + 2h)
b = trench width

When the method of numerical integration is used this formula can be converted into:

$$\Delta h = - \frac{Q_1*(v_1+v_2)}{g*(Q_1+Q_2)} * (\Delta v + v_2 * \frac{\Delta Q}{Q_1}) + s_o*\Delta x - s_f*\Delta x \qquad [2]$$

with Q = discharge = q * x
v = velocity
s_f = friction slope

For dimensioning purposes the friction losses may be neglected if the flow is sub-critical, so $s_f = 0$.

From figure A.1 it can be seen that:

$$\Delta z = s_o * \Delta x - \Delta h$$

so equation [2] can be written as:

$$\Delta z = \frac{Q_1*(v_1+v_2)}{g*(Q_1+Q_2)} * (\Delta v + v_2 * \frac{\Delta Q}{Q_1}) \qquad [3]$$

First, the **maximum bottom slope** must be found for critical flow to occur at the outlet, so that the flow is sub-critical in all other sections.

Critical flow occurs when:

$$v^2/2g = h/2$$

or $Q^2 = h^3 * b^2 * g$,since $Q = v*h*b$

so $h_c = \sqrt[3]{\{Q^2/(b^2*g)\}}$

The maximum discharge under flood conditions Q_{max} **= 34 cumecs** (so q = 1.42 cumecs/m), and the trench width **b = 2.0 m.** (see section 4.2.2.2.),

so h_c **= 3.09 m.**
and v_c **= 5.5 m/s**. at the outlet section.

The critical bottom slope can be found by using equation [3] for the last part Δx of the trench.
For x = 23 m. the values are:

$$Q_1 = q * x = 32.6 \text{ cumecs}$$
$$h_c = \sqrt[3]{\{Q^2/(b^2*g)\}} = 3.00 \text{ m.}$$
$$v_c = Q / (h_c*b) = 5.43 \text{ m/s}$$

Entering these figures in equation [3] gives:

$$\Delta z = 0.16 \text{ m.}$$

and since Δh = (3.09 - 3.00) = 0.09 m. the **critical bottom slope** can be determined:

$$s_o = (\Delta z + \Delta h) / \Delta x$$
$$= (0.16 + 0.09) / 1$$
$$= 0.25$$

So when s_o is no more than 1 : 4 the outlet section can be used as the control section for the computation of the flow profile in the trench.

The procedure is to determine the water surface levels, the depths and the velocities in discrete steps in the upstream direction, using equation [3].

Table A.1 shows the results of these tabulated calculations, when the critical depth and slope are used.

The value of z at x = L is chosen, as is the step size Δx. The value of Δz in column 4 is varied until it is equal to its value in the last column, which contains equation [3]. This iteration occurs at every step Δx in the upstream direction (hence the decreasing x).
The computation must stop before reaching x = 0, since equation [3] is invalid there (Q_o = 0).

TABLE A.1. COMPUTATION OF FLOW PROFILE

x	dx	z0	dz	z	h	A	Q	v	Q1+Q2	v1+v2	dQ	dv	dz
24		0.0		3.09	$\boxed{3.09}$	6.18	34.08	5.51					
23	1	0.3	0.30	3.39	3.14	6.28	32.66	5.20	66.7	10.7	1.42	0.31	0.30
22	1	0.5	0.20	3.59	3.09	6.18	31.24	5.06	63.9	10.3	1.42	0.15	0.20
21	1	0.8	0.18	3.77	3.02	6.04	29.82	4.94	61.1	10.0	1.42	0.12	0.18
20	1	1.0	0.17	3.94	2.94	5.88	28.40	4.83	58.2	9.8	1.42	0.11	0.17
18	2	1.5	0.36	4.30	2.80	5.60	25.56	4.56	54.0	9.4	2.84	0.27	0.36
16	2	2.0	0.37	4.67	2.67	5.34	22.72	4.25	48.3	8.8	2.84	0.31	0.37
14	2	2.5	0.36	5.03	2.53	5.06	19.88	3.93	42.6	8.2	2.84	0.33	0.36
11	3	3.3	0.54	5.57	2.32	4.64	15.62	3.37	35.5	7.3	4.26	0.56	0.54
8	3	4.0	0.49	6.06	2.06	4.12	11.36	2.76	27.0	6.1	4.26	0.61	0.49
5	3	4.8	0.45	6.51	1.76	3.52	7.10	$\boxed{2.02}$	18.5	4.8	4.26	0.74	0.45
2	3	5.5	0.36	6.87	1.37	2.74	2.84	1.04	9.9	3.1	4.26	0.98	0.36
1	1	5.8	0.08	$\boxed{6.95}$	1.20	2.40	1.42	0.59	4.3	1.6	1.42	0.44	0.08

channel width b = 2.00 m.
channel length L = 24.00 m.
specific discharge q = 1.42 cumecs
bottom slope s0 = 0.25

The results show that z_1 is very high (6.95 m) if s_o = 0.25 is applied. Velocities are also higher than necessary. At x = 5 m. the velocity v = 2 m/s, where only 1 m/s is called for (see section 4.2.2.2.).
The trench bottom can therefore be set at a much smaller slope. Table A.2 shows that a slope of 1 : 10 is the minimum to keep the water moving at sufficient speed.

TABLE A.2. COMPUTATION OF FLOW PROFILE

x	dx	z0	dz	z	h	A	Q	v	Q1+Q2	v1+v2	dQ	dv	dz
24		0.0		3.09	$\boxed{3.09}$	6.18	34.08	5.51					
23	1	0.1	0.62	3.71	3.61	7.22	32.66	4.52	66.7	10.0	1.42	0.99	0.62
22	1	0.2	0.25	3.96	3.76	7.52	31.24	4.15	63.9	8.7	1.42	0.37	0.25
21	1	0.3	0.20	4.16	3.86	7.72	29.82	3.86	61.1	8.0	1.42	0.29	0.20
20	1	0.4	0.16	4.32	3.92	7.84	28.40	3.62	58.2	7.5	1.42	0.24	0.16
18	2	0.6	0.27	4.59	3.99	7.98	25.56	3.20	54.0	6.8	2.84	0.42	0.27
16	2	0.8	0.23	4.82	4.02	8.04	22.72	2.83	48.3	6.0	2.84	0.38	0.23
14	2	1.0	0.19	5.01	4.01	8.02	19.88	2.48	42.6	5.3	2.84	0.35	0.19
11	3	1.3	0.24	5.25	3.95	7.90	15.62	1.98	35.5	4.5	4.26	0.50	0.24
8	3	1.6	0.18	5.43	3.83	7.66	11.36	1.48	27.0	3.5	4.26	0.49	0.18
5	3	1.9	0.13	5.56	3.66	7.32	7.10	$\boxed{0.97}$	18.5	2.5	4.26	0.51	0.14
2	3	2.2	0.08	5.64	3.44	6.88	2.84	0.41	9.9	1.4	4.26	0.56	0.08
1	1	2.3	0.01	$\boxed{5.65}$	3.35	6.70	1.42	0.21	4.3	0.6	1.42	0.20	0.01

channel width b = 2.00 m.
channel length L = 24.00 m.
specific discharge q = 1.42 cumecs
bottom slope s0 = 0.10

The **depth of the trench** must be taken at **5.70 m.**, to keep the grid clear of the water surface.
To check what would happen if the capacity of the flushing pipes turns out to be smaller than expected, the computation is rerun with higher outlet depths. By trial and error the upper limit was found to be h_{24} = 3.60 m. Table A.3 shows that then the water surface almost reaches the grid and the velocities will still be sufficient.

TABLE A.3. COMPUTATION OF FLOW PROFILE

x	dx	z0	dz	z	h	A	Q	v	Q1+Q2	v1+v2	dQ	dv	dz
24		0.0		3.60	$\boxed{3.60}$	7.20	34.08	4.73					
23	1	0.1	0.28	3.88	3.78	7.56	32.66	4.32	66.7	9.1	1.42	0.41	0.28
22	1	0.2	0.20	4.08	3.88	7.76	31.24	4.03	63.9	8.3	1.42	0.29	0.20
21	1	0.3	0.17	4.25	3.95	7.90	29.82	3.77	61.1	7.8	1.42	0.25	0.17
20	1	0.4	0.15	4.40	4.00	8.00	28.40	3.55	58.2	7.3	1.42	0.22	0.15
18	2	0.6	0.26	4.66	4.06	8.12	25.56	3.15	54.0	6.7	2.84	0.40	0.26
16	2	0.8	0.21	4.87	4.07	8.14	22.72	2.79	48.3	5.9	2.84	0.36	0.21
14	2	1.0	0.18	5.05	4.05	8.10	19.88	2.45	42.6	5.2	2.84	0.34	0.18
11	3	1.3	0.23	5.28	3.98	7.96	15.62	1.96	35.5	4.4	4.26	0.49	0.23
8	3	1.6	0.18	5.46	3.86	7.72	11.36	1.47	27.0	3.4	4.26	0.49	0.18
5	3	1.9	0.13	5.59	3.69	7.38	7.10	$\boxed{0.96}$	18.5	2.4	4.26	0.51	0.13
2	3	2.2	0.08	5.67	3.47	6.94	2.84	0.41	9.9	1.4	4.26	0.55	0.08
1	1	2.3	0.01	$\boxed{5.68}$	3.38	6.76	1.42	0.21	4.3	0.6	1.42	0.20	0.01

channel width b = 2.00 m.
channel length L = 24.00 m.
specific discharge q = 1.42 cumecs
bottom slope s0 = 0.10

APPENDIX B. COMPUTATION OF FLOW IN FLUSHING CHANNELS

The lateral flushing channels in the floor of the desilting basin catch the lower layers of the flow, with the settled sediments and transport it sideways into the sluiceway (figure B.1).

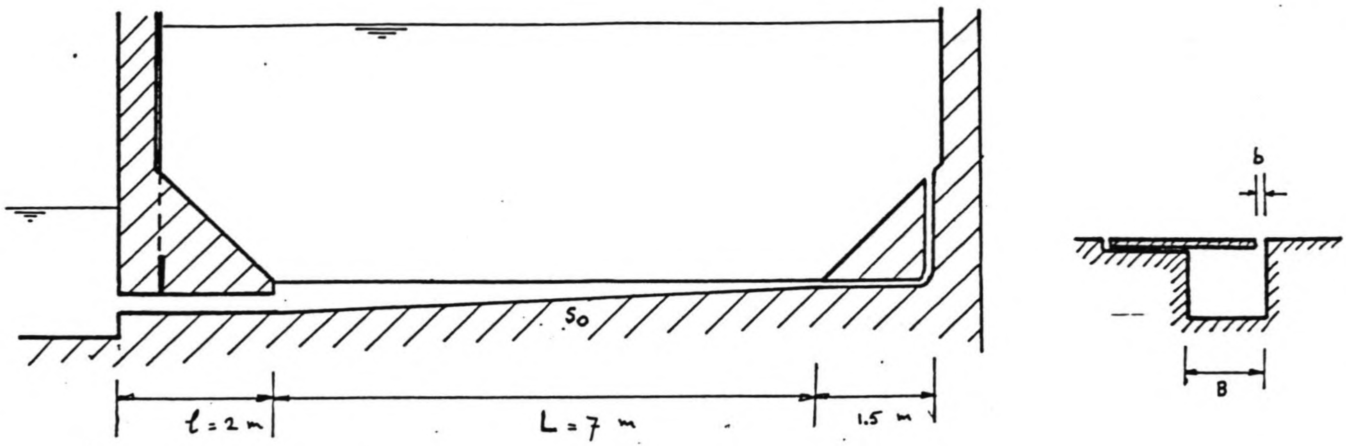


FIGURE B.1 Flushing channel sections

The discharge entering through the slot is dependant on the difference in pressure inside and outside the channel (figure B.2):

$$q = \mu * b * \sqrt{2g * (H-p)} \qquad [1]$$

with q = specific discharge
b = slot width
μ = contraction coefficient
H = water depth in basin
p = piezometric head in channel

This formula is based on the energy equation of Bernouilli. The contraction coefficient of an orifice with a straight face and sharp edges is ± 0.60 (see also section 6.3.3.1). In this case the edges of the tiles will be rough and irregular, so the coefficient is estimated to be μ = 0.58.

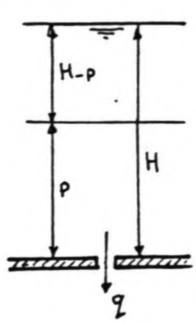


FIGURE B.2

Specific discharge through slot

Continuity of the flow in the channel dictates that (figure B.3):

$$Q_1 = Q_0 + q*\Delta x \qquad [2]$$

or $(v*h*B)_1 = (v*h*B)_0 + q*\Delta x$ with h = channel depth
B = channel width

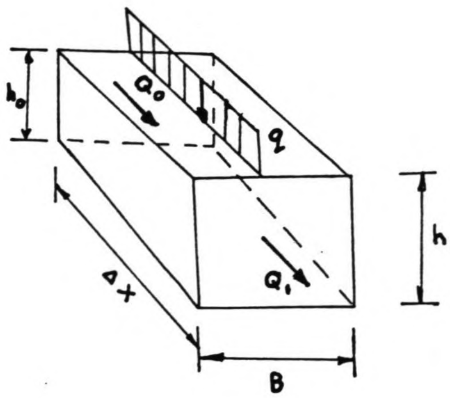


FIGURE B.3

Continuity of flow

The flow in the channel can also be described with the **momentum** equation (figure B.4):

$$P_0 - P_1 - F = \rho_w * (Q_1*v_1 - Q_0*v_0) \qquad [3]$$

with P = hydrostatic force
F = friction force
ρ_w = specific weight of water

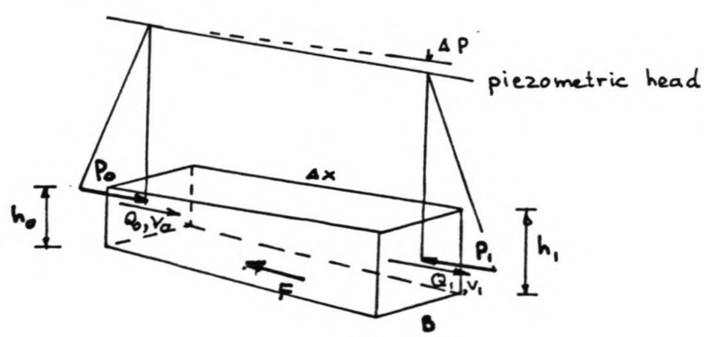


FIGURE B.4

Momentum principle

Since $P = (p + h/2)*g*\rho_w*B*h$
and $Q = v*h*B$

equation [3] can be written as:

$$(p_0+h_0/2)g\rho_w h_0 B - (p_1+h_1/2)g\rho_w h_1 B = \rho_w(v_1^2 h_1 B - v_0^2 h_0 B) + F$$

or $$\Delta p = (h_0 v_0^2 - h_1 v_1^2)/g + (h_0 - h_1)/2 + \Delta H_f \qquad [4]$$

with Δp = loss of piezometric head over Δx
ΔH_f = friction loss over Δx.

The friction losses can be estimated with Manning's formula:

$$v = M * \sqrt{s_f} * R^{0.67}$$
and $\Delta H_f = \Delta x * s_f$

with M = roughness coefficient (= 60 for rough concrete)
R = hydraulic radius of channel (= B*h /2*(B+h))

The piezometric line can now be computed, if the boundary values are known:

H_b = water depth in basin
H_s = water depth in sluiceway
ΔH_p = losses in the pipe leading to the sluiceway
Q_o = base discharge at x = 0
v_{min} = minimum velocity for sediment transport

The channel length is set at L = 7 m., and the other dimensions: b, B, h_o and s_o are varied to satisfy the design criteria (see section 6.2.2).

The pipe losses must be estimated first. They consist of:

- entrance losses $\Delta H_e = 0.1*(v^2/2g)$ (see fig.4.18)
- friction losses $\Delta H_f = 1*(v^2/(M*R^{0.67}))^2$

Table B.1 shows the tabulated computation. For each step Δx the Δp in column 5 is varied until it is identical to the Δp in the last column, which contains equation [4]. The velocity in line 1 is varied until the minimum v = 1 m/s occurs at x = 0.

TABLE B.1. COMPUTATION OF FLOW IN FLUSHING CHANNEL

X	dX	h	p	dp	(H-p)	q	Q	v	R	dp
7.00		0.30	0.54		2.30		0.26	4.39		
6.00	1.00	0.27	0.90	0.36	1.94	0.04	0.22	4.04	0.06	0.36
5.00	1.00	0.24	1.22	0.32	1.62	0.04	0.18	3.69	0.05	0.32
4.00	1.00	0.21	1.49	0.28	1.35	0.04	0.14	3.32	0.05	0.28
3.00	1.00	0.18	1.73	0.24	1.11	0.03	0.11	2.93	0.05	0.24
2.00	1.00	0.15	1.92	0.19	0.92	0.03	0.07	2.48	0.04	0.19
1.00	1.00	0.12	2.07	0.15	0.77	0.03	0.05	1.92	0.04	0.15
0.75	0.25	0.11	2.10	0.03	0.74	0.01	0.04	1.75	0.04	0.03
0.50	0.25	0.11	2.13	0.03	0.71	0.01	0.03	1.56	0.03	0.03
0.25	0.25	0.10	2.15	0.02	0.69	0.01	0.03	1.35	0.03	0.02
0.00	0.25	0.10	2.16	0.01	0.68	0.01	0.02	1.00	0.03	0.01

channel length	L =	7.00	basin water level	Hb =	2.84
width	B =	0.20	sluiceway water level	Hs =	0.00
slope	s0 =	0.03	pipe losses	Δ Hp =	0.54
slot width	b =	0.012	Manning's	M =	60
contr. coeff.	μ =	0.58	base discharge	Q0 =	0.02

The pipe losses are based on a pipe length l = 2 m., diameter D = 0.25 m., and v = 5 m/s (in the pipe).
The hydraulic conditions are:

- minimum river level: Q_r = 30 cumecs,
  H_b = 2.84 m. (section 6.2.1.2)
- sluiceway water level: 734.90 m.EL.

Table B.1 shows that the design criteria can be met under these conditions.

To assess the sensitivity of the system to changes in the sluiceway water level the computation has also been done for H_s = -0.20 m. and H_s = 1.20 m. The first level occurs when there is a free jet out of the flushing pipe, so when the sluiceway water level drops below the pipe outlet level (734.70 m.). The last occurs when the hydraulic jump in the sluiceway is drowned out. The results of these computations are shown in tables B.2 and B.3. They show that the system is not very sensitive to these changes.

It can be concluded from these computations that for Q_r = 30 cumecs the sluiceway gates should be operated to create a water level of ± 735.0 m. or less. Higher levels are no problem for short periods however.

TABLE B.2. COMPUTATION OF FLOW IN FLUSHING CHANNEL

X	dX	h	p	dp	(H-p)	q	Q	v	R	dp
7.00		0.30	0.37		2.47		$\boxed{0.27}$	4.52		
6.00	1.00	0.27	0.75	0.38	2.09	0.05	0.22	4.16	0.06	0.38
5.00	1.00	0.24	1.09	0.34	1.75	0.04	0.18	3.79	0.05	0.34
4.00	1.00	0.21	1.38	0.29	1.46	0.04	0.14	3.40	0.05	0.29
3.00	1.00	0.18	1.63	0.25	1.21	0.04	0.11	2.98	0.05	0.25
2.00	1.00	0.15	1.83	0.20	1.01	0.03	0.07	2.49	0.04	0.20
1.00	1.00	0.12	1.98	0.15	0.86	0.03	0.04	1.87	0.04	0.15
0.75	0.25	0.11	2.01	0.03	0.83	0.01	0.04	1.68	0.04	0.03
0.50	0.25	0.11	2.04	0.03	0.80	0.01	0.03	1.47	0.03	0.03
0.25	0.25	0.10	2.06	0.02	0.78	0.01	0.02	1.23	0.03	0.02
0.00	0.25	0.10	2.07	0.01	0.77	0.01	0.02	0.86	0.03	0.01

channel length	L =	7.00		basin water level	Hb =	2.84
width	B =	0.20		sluiceway water level	Hs =	-0.20
slope	s0 =	0.03		pipe losses	ΔHp =	0.57
slot width	b =	0.012		Manning's	M =	60
contr. coeff.	μ =	0.58		base discharge	Q0 =	0.02

TABLE B.3. COMPUTATION OF FLOW IN FLUSHING CHANNEL

X	dX	h	p	dp	(H-p)	q	Q	v	R	dp
7.00		0.30	1.50		1.34		0.20	3.30		
6.00	1.00	0.27	1.71	0.21	1.13	0.03	0.16	3.03	0.06	0.21
5.00	1.00	0.24	1.90	0.19	0.94	0.03	0.13	2.76	0.05	0.19
4.00	1.00	0.21	2.06	0.16	0.78	0.03	0.10	2.47	0.05	0.16
3.00	1.00	0.18	2.20	0.14	0.64	0.03	0.08	2.17	0.05	0.14
2.00	1.00	0.15	2.31	0.11	0.53	0.02	0.05	1.82	0.04	0.11
1.00	1.00	0.12	2.40	0.09	0.44	0.02	0.03	1.38	0.04	0.09
0.75	0.25	0.11	2.42	0.02	0.42	0.01	0.03	1.25	0.04	0.02
0.50	0.25	0.11	2.44	0.02	0.40	0.00	0.02	1.10	0.03	0.02
0.25	0.25	0.10	2.45	0.01	0.39	0.00	0.02	$\boxed{0.94}$	0.03	0.01
0.00	0.25	0.10	2.46	0.01	0.38	0.00	0.01	$\boxed{0.68}$	0.03	0.01

channel length	L =	7.00	basin water level	Hb =	2.84
width	B =	0.20	sluiceway water level	Hs =	1.20
slope	s0 =	0.03	pipe losses	Δ Hp =	0.30
slot width	b =	0.012	Manning's	M =	60
contr. coeff.	μ =	0.58	base discharge	Q0 =	0.01

The same calculations have been done to see whether the criteria are met under the design flood conditions:

Q_r = 500 cumecs,

H_b = 2.97 m. (see section 6.2.1.4).

In this case the flushing discharge can be higher:

Q_f = 4.7 cumecs, or 0.59 cumecs per channel.

Table B.4 shows that this is clearly not the limiting factor, so it is quite acceptable for the sluiceway water level to drop below the pipe outlet level.

TABLE B.4. COMPUTATION OF FLOW IN FLUSHING CHANNEL

X	dX	h	p	dp	(H-p)	q	Q	v	R	dp
7.00		0.30	0.40		2.57		$\boxed{0.28}$	4.63		
6.00	1.00	0.27	0.80	0.40	2.17	0.05	0.23	4.27	0.06	0.40
5.00	1.00	0.24	1.15	0.35	1.82	0.04	0.19	3.89	0.05	0.35
4.00	1.00	0.21	1.46	0.31	1.51	0.04	0.15	3.50	0.05	0.31
3.00	1.00	0.18	1.72	0.26	1.25	0.04	0.11	3.08	0.05	0.26
2.00	1.00	0.15	1.93	0.21	1.04	0.03	0.08	2.59	0.04	0.21
1.00	1.00	0.12	2.09	0.16	0.88	0.03	0.05	1.98	0.04	0.16
0.75	0.25	0.11	2.12	0.03	0.85	0.01	0.04	1.80	0.04	0.03
0.50	0.25	0.11	2.15	0.03	0.82	0.01	0.03	1.59	0.03	0.03
0.25	0.25	0.10	2.17	0.02	0.80	0.01	0.03	1.35	0.03	0.02
0.00	0.25	0.10	2.18	0.01	0.79	0.01	0.02	0.98	0.03	0.01

channel length	L =	7.00	basin water level	Hb =	2.97
width	B =	0.20	sluiceway water level	Hs =	-0.20
slope	s0 =	0.03	pipe losses	Δ Hp =	0.60
slot width	b =	0.012	Manning's	M =	60
contr. coeff.	μ =	0.58	base discharge	Q0 =	0.02

Table B.5 shows that in the first 0.75 m. of the trench the velocities drop below v = 1 m/s, so there is a clogging danger there.

TABLE B.5. COMPUTATION OF FLOW IN FLUSHING CHANNEL

X	dX	h	p	dp	(H-p)	q	Q	v	R	dp
7.00		0.30	2.01		0.96		0.17	2.75		
6.00	1.00	0.27	2.16	0.15	0.81	0.03	0.14	2.52	0.06	0.15
5.00	1.00	0.24	2.29	0.13	0.68	0.03	0.11	2.28	0.05	0.13
4.00	1.00	0.21	2.41	0.12	0.56	0.02	0.09	2.03	0.05	0.12
3.00	1.00	0.18	2.51	0.10	0.46	0.02	0.06	1.76	0.05	0.10
2.00	1.00	0.15	2.59	0.08	0.38	0.02	0.04	1.44	0.04	0.08
1.00	1.00	0.12	2.65	0.06	0.32	0.02	0.02	1.04	0.04	0.06
0.75	0.25	0.11	2.66	0.01	0.31	0.00	0.02	0.91	0.04	0.01
0.50	0.25	0.11	2.67	0.01	0.30	0.00	0.02	0.77	0.03	0.01
0.25	0.25	0.10	2.68	0.01	0.29	0.00	0.01	0.62	0.03	0.01
0.00	0.25	0.10	2.68	0.00	0.29	0.00	0.01	0.40	0.03	0.00

channel length	L = 7.00		basin water level	Hb = 2.97
width	B = 0.20		sluiceway water level	Hs = 1.80
slope	s0 = 0.03		pipe losses	Δ Hp = 0.21
slot width	b = 0.012		Manning's	M = 60
contr. coeff.	μ = 0.58		base discharge	Q0 = 0.01

APPENDIX C. HYDRAULIC TRANSPORT OF SEDIMENTS

The transport capacity of the flushing channels in the floor of the desilting basin should be sufficient to eject the full sediment load of the intake discharge back into the river.
The sediment load of the river is estimated to be 2500 m^3 per year per km^2 watershed, of which 15 % is bed load. Since the watershed of the Jhimruk Khola above the project site is 645 km^2 this amounts to $1.6*10^6$ m^3 (or $4.2*10^9$ kg) per year. This will probably all be concentrated in the monsoon period, since the river is clear in the dry season.
The mean total discharge in these months (july - oct.) is $6.4 * 10^8$ m^3 (see fig. 2.3), so the average sediment load is 1.6/6.4 = 0.25 %, or 6.5 kg/m^3.

The amount of sediment entering the intake at maximum flow (Q_t+Q_f=1.3*6.9=9 cumecs) can therefore be estimeted to be:

$$0.85 * 0.0025 * Q_i = 0.019 \text{ cumecs} \quad (\text{or } 0.21 \%)$$

assuming the 15 % bed load is diverted away from the intake windows.

The transport capacity of the flushing pipes is taken as indicator, the channels themselves having about the same hydraulic diameter in the last section.
This capacity can be assessed using the results of experiments done by Führböter [litt.2].
Figure C.1 shows that the critical velocity is about 4 m/s for all sediment concentrations C_T bigger than 10 %. This figure is valid for a pipe diameter D = 300 mm., and a mean grain diameter $d_m \approx 1$ mm. (▪—▪ line).

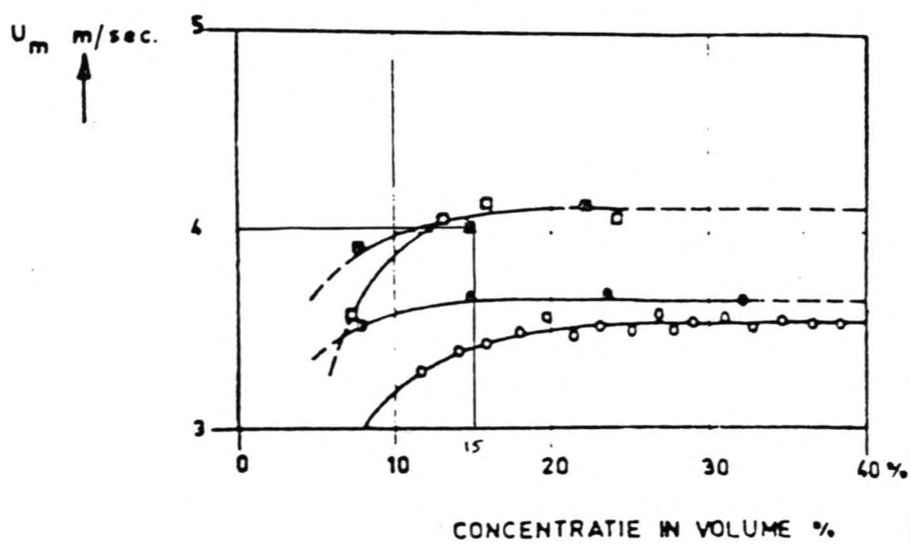


FIGURE C.1 (source: litt.2)

Critical velocity for sediment transport in pipelines

Figure C.2 shows the influence of the pipe diameter D on the critical velocity V_{cr}. In the case of the flushing pipes D = 250 mm, so the V_{cr} of figure C.1 must be multiplied by a factor {D=25}/{D=30} = 0.70/0.75 = 0.93.

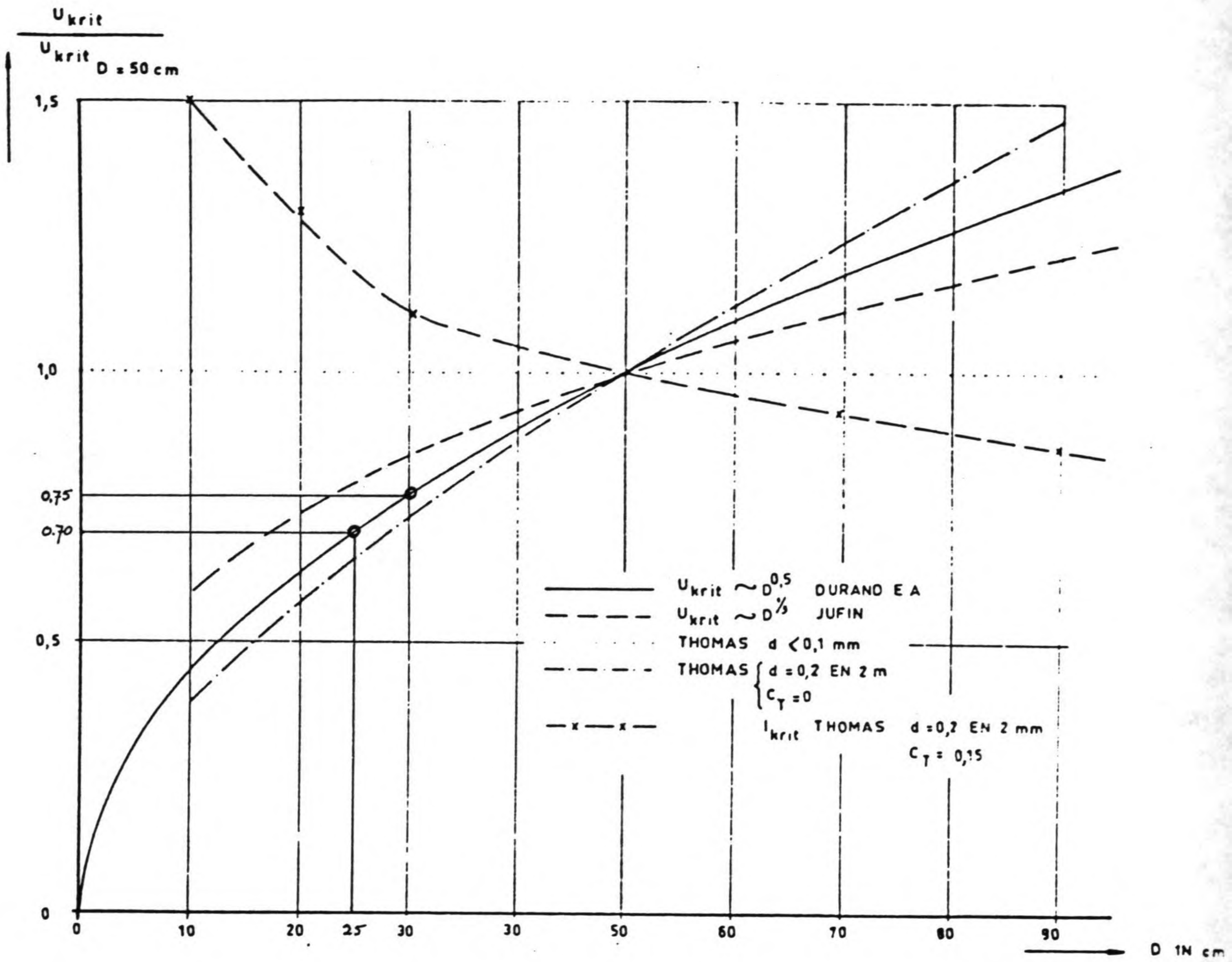


FIGURE C.2 (source: litt.2)

Influence of pipe diameter on critical velocity

Figure C.3 shows the influence of the sediment concentration C_T on the critical velocity.
If the flushing discharge is ± 30 % of the turbine discharge the sediment concentration will be:

$$C_T = 0.21 * 1.3/ 0.3 \approx 0.9 \%$$

This is an average however, and the concentration in one channel could be several times higher than this, depending on the sieve curve of the sediment, since the heaviest particles will settle in the first part and the lightest in the last part of the basin. The coarsest sediment will occur in the first channel, thus the limiting case.
Assumed C_T = 3 %.

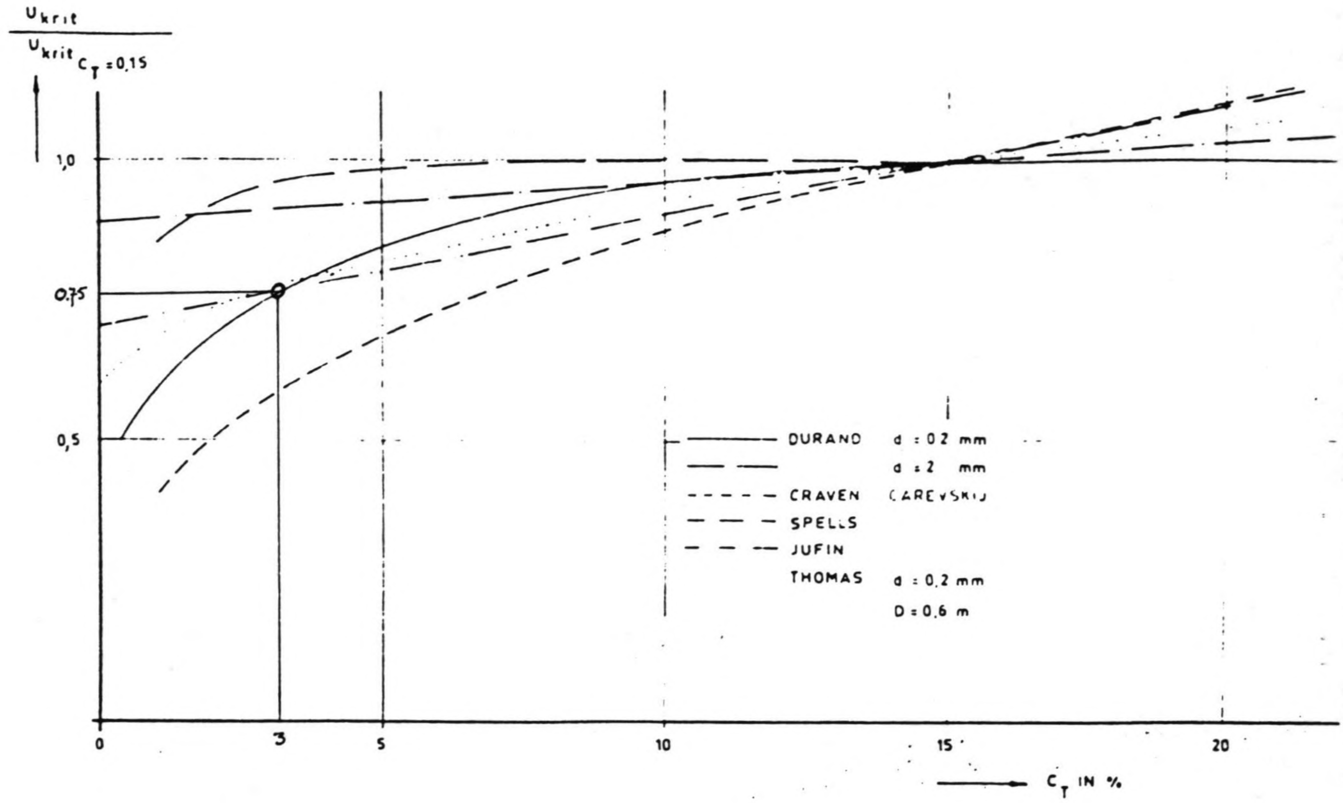


FIGURE C.3 Influence of C_T on V_{cr} (source: litt.2)

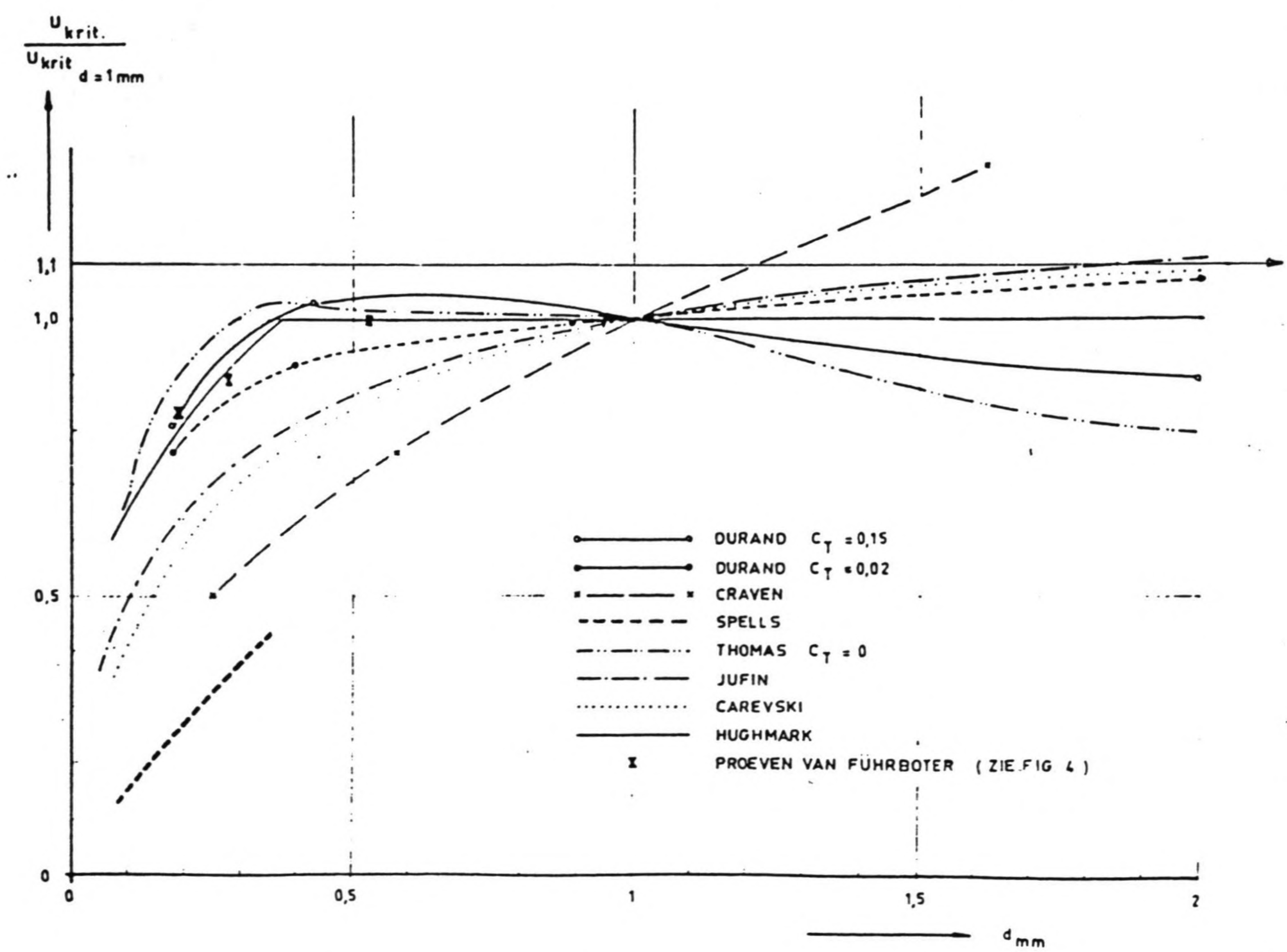


FIGURE C.4 Influence of d_m on V_{cr} (source: litt.2)

Figure C.4 shows the influence of the mean grain diameter. The maximum diameter expected to enter the intake, and thus reach the first channel, is $d_{max} \approx$ 10 mm, so a mean diameter of $d_m \approx$ 5 mm. is a safe guess.
The graph is extrapolated to the right, and a factor of 1.1 is assumed to be reasonable

To arrive at the critical velocity for the limiting conditions the value found in figure C.1 must be multiplied by the factors found in figures C.2 to C.4:

$$V_{cr} = 4 * (0.70/0.75) * 0.75 * 1.1 = 3.1 \text{ m/s}$$

Such a velocity will lead to a discharge of:

$$\begin{aligned} Q_1 &= v_1 * 1/4 * \pi * D^2 \\ &= 3.1 * 0.25 * 3.14 * 0.25^2 \\ &= 0.15 \text{ cumecs} \end{aligned}$$

This is the minimum for the first flushing pipe; the others can have lower velocities. If we assume it to be the average value for all eight channels the total necessary flushing discharge would be

$$\Sigma Q = 8 * 0.15 = 1.2 \text{ cumecs}$$

which is far less than the assumed 30 % of the turbine discharge (= 2.1 cumecs), so the flushing discharge is clearly sufficient to create the necessary sediment transport capacity.

APPENDIX D. COMPUTATION OF FLOW IN SLUICEWAY

The formulas that are to be used for this computation have already been presented in section 6.3.2, and are only mentioned here:

gate discharge:

$$Q = B * c_o * w * \sqrt{(19.6 * H_o)}$$
$$c_o = \mu / \sqrt{(1 + \mu * w / H_o)}$$
$$\mu = 1 - 0.75*(\theta/90°) + 0.36*(\theta/90°)$$

friction slope:

$$s_f = (v / M*R^{0.67})^2$$

hydraulic jump:

$$h_2/h_1 = 0.5 * \{\sqrt{(1 + 8*F_1^2)} - 1\}$$
$$F = v / \sqrt{(g * h)}$$

This formula for the hydraulic jump is valid for a horizontal bottom (s_o = 0). For sloping channels the ratio of sequent depth (y_2) to initial depth (y_1) is higher. Figure D.1 gives the graphs that can be used for design purposes [litt.4].

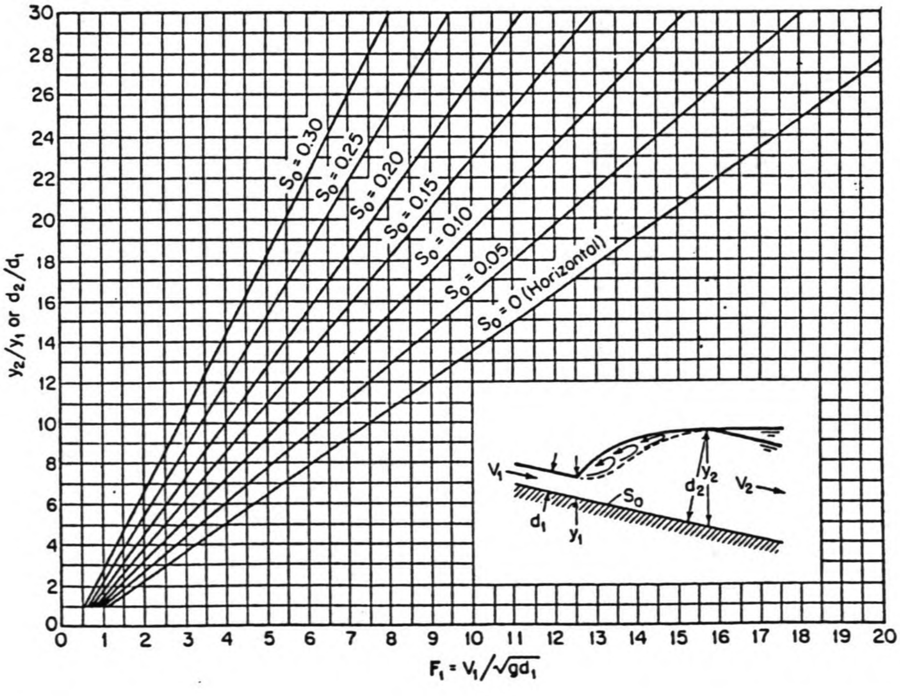


FIGURE D.1 Relations between F and y_2/y_1 (source: litt.4)

The length of the hydraulic jump (L) has also been established in relation to the Froude number and the sequent depth. Figure D.2 shows these relationships for several channel slopes.

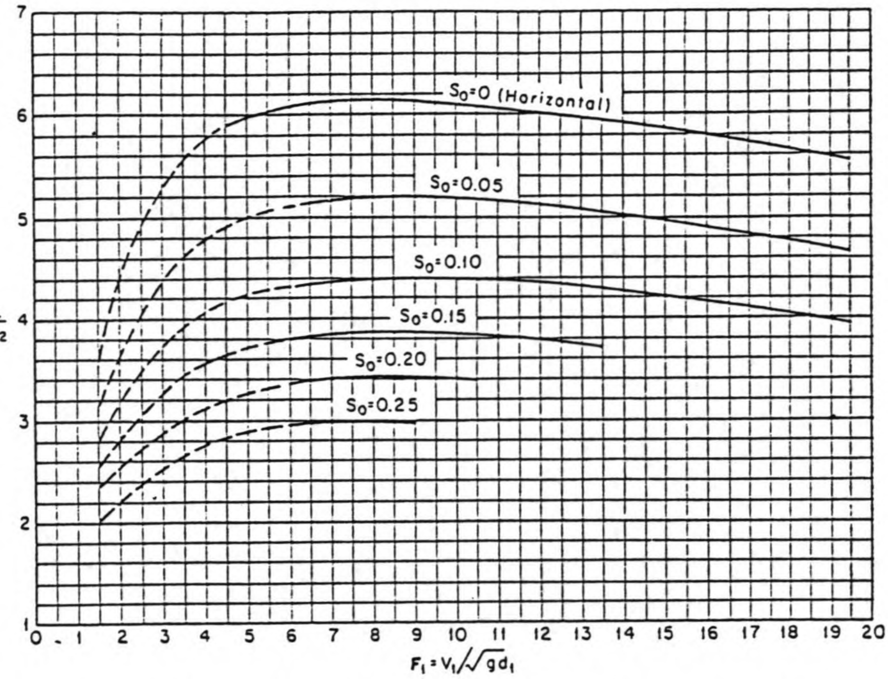


FIGURE D.2 Relationships of F and L/y_2 (source: litt.4)

In the first part behind the gates the flow must be computed in the downstream direction, because the flow is supercritical there. After the jump it is subcritical, so the computation should be in the upstream direction.

The boundary values are:

- the backwater level of the river, and
- the gate opening (which give the discharge and the initial depth)
- the tailwater level of the river (which gives the sequent depth)

The design parameters that can be varied are:

- gate level
- gate size
- sluiceway width
- sluiceway bottom slope

The first two parameters have been established in section 6.3.3.1. Now the other two should be fixed, in relation with the gate operation described in section 6.3.3.2, in a way that satisfies the criteria [1], [2], [4] and [5].

The computation of flow in the sluiceway involves many trial and error iterations, and is therefore suitable for computer application. With the use of Lotus 1-2-3 a spread-sheet program was written (GATEFLOW), which is presented in table D.1.

The **input** of physical data is shown below the table (column 2). The necessary discharge constants are calculated from those data, and shown below column 7. Below column 12 the calculated sluiceway discarge Q is shown, and the calculation steps Δx_1 (first five steps) and Δx_2 (last five steps) must be put in.

The output of flow parameters is shown in the table itself. The columns contain the hydraulic formulas.

Column 5: friction slope over step Δx (Mannings formula)
7: decrease of energy head (above bottom level)
8: increase of depth (guess)
9: decrease of velocity (result of 7 and 8)
10-12: resulting parameters
13: check of guessed value of Δh
14: new guess of Δh

GATEFLOW iterates this computation of 8 to 14 through the "macro's" \P, \Q and \R. These routines are shown below the table. The iteration is stopped as soon as the error in $Q < 0.01$ cumecs, or when the process does not converge to a solution.

When the approximate solution has been found the sequent depth and hydraulic jump length are computed.

Column 15: Froude number at x
17: sequent depth according to figure D.1
18: jump length according to figure D.2
19: actual depth at x + L

When the actual depth is smaller then the computed sequent depth the hydraulic jump will not occur at x. The process continues on the next line with another step Δx, until $y_2' < y_2$.

#P and #R are iteration counters, and the discharge error (Q - Q') is shown for the last computed line.

Table D.1 gives the case of the river flood (Q_r = 500 cumecs, see table 2.3), and with the gates completely lifted. The gate dimensions are the ones arrived at in section 6.3.3.1. The sluiceway bottom must be horizontal, because of the minimum level defined by criterion [4]: 734.0 m.EL (see section 6.3.1.).

The results show that the jump will **not** occur within a reasonable distance from the gates, so criterion [5] can not be satisfied under these conditions.

TABLE D.1 COMPUTATION OF FLOW IN SLUICEWAY

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
x	h	R	v	s	E	dE[cm]	dh[cm]	dv	h'	v'	Q'	Q-Q'	dh'	Fr	V-hd	y2'	L	y2	x
2.95	1.10	0.76	8.4	0.028	4.679	5.65	0.74	0.06	1.11	8.4	38.83	0.004	0.75	2.54	3.56	3.46	18.19	2.67	4.95
4.95	1.11	0.77	8.4	0.028	4.638	5.54	0.75	0.06	1.11	8.3	38.83	0.000	0.75	2.51	3.52	3.44	17.97	2.67	6.95
6.95	1.11	0.77	8.3	0.027	4.599	5.43	0.75	0.06	1.12	8.3	38.83	0.000	0.75	2.49	3.47	3.42	17.74	2.67	8.95
8.95	1.12	0.77	6.9	0.019	3.547	3.76	0.70	0.04	1.13	6.9	38.83	0.000	0.70	2.07	2.42	2.79	12.58	2.67	10.95
10.95	1.13	0.78	6.9	0.018	3.524	3.69	0.70	0.04	1.13	6.8	38.83	0.000	0.70	2.05	2.39	2.77	12.44	2.67	12.95
12.95	1.13	0.78	6.8	0.018	3.502	3.62	0.70	0.04	1.14	6.8	38.83	0.000	0.70	2.03	2.36	2.76	12.30	2.67	14.95
14.95	1.14	0.78	6.8	0.018	3.480	3.56	0.70	0.04	1.15	6.8	38.83	0.000	0.70	2.02	2.33	2.75	12.16	2.67	16.95
16.95	1.15	0.79	6.8	0.017	3.458	3.50	0.69	0.04	1.16	6.7	38.83	0.000	0.69	2.00	2.31	2.74	12.02	2.67	18.95
18.95	1.16	0.79	6.7	0.017	3.438	3.44	0.69	0.04	1.16	6.7	38.83	0.000	0.69	1.98	2.28	2.72	11.89	2.67	20.95
20.95	1.16	0.79	6.7	0.017	3.417	3.38	0.69	0.04	1.17	6.6	38.83	0.000	0.69	1.96	2.25	$\boxed{2.71}$	11.76	$\boxed{2.67}$	22.95

M =	60	theta=	1.57	Q =	38.83
a =	1.80	μ =	0.61		
r =	3.00	c0=	0.55		
s0 =	0.000	H0=	4.50	dx1=	2.00
BWL=	738.50			dx2=	2.00
TWL=	736.67				
w =	1.80				
b =	2.10				
B =	5.00				
SBL=	734.00				

```
\P  {blank T22}{FOR Q20,1,10,1,\R}

      #P =    11

      #R =     1

    error in Q=  0.000

\Q  {windowsoff}{right 4}/rv~{left 6}~
    {recalc CONT}{left 1}/rv~R24~
    {left 3}{windowson}{windowsoff}
    {look T22}{if T22="q"}{quit}
    {if @ABS(R24)<0.01}{forbreak}
    {if @isERR(R24)}{quit}
    {if @ABS(R24)>0.5*L18}{quit}

\R  {FOR Q22,1,10,1,\Q}{recalc EQUI,(V1>1),3}
    {right 7}/rv~U11~{right 2}/rv~U13~{if (U11-U13)<0}{forbreak}
    {left 9}{down}{recalc B7..V16}{windowson}{if @ABS(R24)>0.01}{quit}
```

Decreasing the gate opening could help, because the smaller initial depth y_1 would lead to a smaller sequent depth y_2'. The gate discharge would also be smaller though, which would result in sediment transport problems (see section 6.3.3.1).

A sloping sluiceway was tried, with levels decreasing from 734.5 m.EL at the gates to 734.0 m.EL at x = 30 m. Table D.2 gives the results, which show that the slope does not help at all. The length of the jump is smaller, but the higher Froude number and y_2/y_1 ratio more than offset this effect.

The conclusion must be that **the necessary length of the sluiceway can not be kept within reasonable limits, without energy dissipation or jump fixation measures.** This is worked out in section 6.3.3.2.

TABLE D.2 COMPUTATION OF FLOW IN SLUICEWAY

1	2	4	7	8	9	12	13	14	15	16	17	18	19	20
x	h	v	dE[cm]	dh[cm]	dv	Q'	Q-Q'	dh'	Fr	V-hd	y2'	L	y2	x
2.95	1.10	7.8	1.50	0.20	0.02	36.16	0.008	0.21	2.38	3.12	3.38	15.08	2.51	4.95
4.95	1.10	7.8	1.48	0.20	0.02	36.16	0.008	0.20	2.38	3.11	3.38	15.03	2.54	6.95
6.95	1.10	7.8	1.45	0.20	0.02	36.16	0.009	0.20	2.37	3.10	3.37	14.99	2.58	8.95
8.95	1.10	6.6	0.00	0.00	0.00	36.17	0.000	0.00	1.99	2.19	2.75	10.95	2.54	10.95
10.95	1.10	6.6	0.00	0.00	0.00	36.17	0.000	0.00	1.99	2.19	2.75	10.95	2.58	12.95
12.95	1.10	6.6	0.00	0.00	0.00	36.17	0.000	0.00	1.99	2.19	2.75	10.95	2.61	14.95
14.95	1.10	6.6	0.00	0.00	0.00	36.17	0.000	0.00	1.99	2.19	2.75	10.95	2.64	16.95
16.95	1.10	6.6	0.00	0.00	0.00	36.17	0.000	0.00	1.99	2.19	2.75	10.95	2.67	18.95
18.95	1.10	6.6	0.00	0.00	0.00	36.17	0.000	0.00	1.99	2.19	2.75	10.95	2.67	20.95
20.95	1.10	6.6	0.00	0.00	0.00	36.17	0.000	0.00	1.99	2.19	$\boxed{2.75}$	10.95	$\boxed{2.67}$	22.95

M = 60	theta= 1.57	Q = 36.17
a = 1.80	μ = 0.61	
r = 3.00	c0= 0.54	
s0 = 0.017	H0= 4.00	dx1= 2.00
BWL= 738.50		dx2= 2.00
TWL= 736.67		
w = 1.80		
b = 2.10		
B = 5.00		
SBL= 734.50		

APPENDIX E. REINFORCED CONCRETE SECTIONS

E.1. Some rough reinforcement calculations have been done on crucial sections of the structure, to check the assumed dimensions. A low reinforcement percentage is aimed for, because steel is an expensive commodity in Nepal.
Figure E.1 shows which sections have been checked.

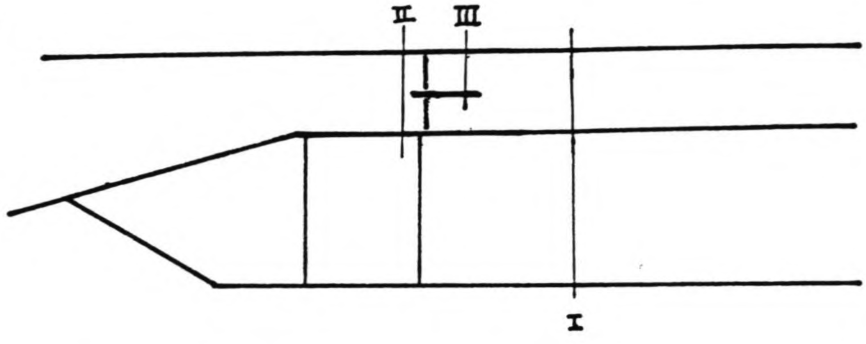


FIGURE E.1. Reinforced concrete sections

For the calculations the load assumptions have been kept simple: only the main loads, without dynamic effects, and with simple schematisations of the structures. More extensive calculations will have to be done in the detailed design stage.

For most loads a safety factor of 1.7 is chosen. It could be argued that in the (economic) context of rural Nepal a lower margin is acceptable. There are considerable uncertainties to be taken in account however:

- unreliable quality of concrete and steel
- less than perfect building practices
- occasional extra forces due to boulders, floating trees, landslides, etc.
- dynamic effects.

Especially where the forces on the sluiceway gates are concerned the dynamic effects can be considerable. A safety factor of 2.0 is adopted for these loads.

For concrete and steel quality relatively low values have been assumed: C 17.5 and Fe 220. The quality of the supplied cement in Nepal often falls short of the specifications, and deficient management of storage often leads to further deterioration. Low quality steel can be bought with local currency in India, but for higher qualities hard currencies are required.

For determining the reinforcement tables E.1 and E.2, and figure E.2 are used.

buiging zonder normaalkracht
bij rechthoekige doorsneden

B 17,5

$\frac{M_u}{bh^2}$	FeB 220		FeB 400		FeB 500		k_x	k_z
	k_a	ω_0	k_a	ω_0	k_a	ω_0		
100	0,219	0,05	0,398	0,03	0,498	0,02	0,011	0,996
200	0,218	0,09	0,397	0,05	0,496	0,04	0,022	0,992
300	0,217	0,14	0,395	0,08	0,494	0,06	0,034	0,988
400	0,216	0,18	0,394	0,10	0,492	0,08	0,045	0,984
500	0,216	0,23	0,392	0,13	0,490	0,10	0,057	0,980
600	0,215	0,28	0,390	0,15	0,488	0,12	0,068	0,976
700	0,214	0,33	0,389	0,18	0,486	0,14	0,080	0,972
800	0,213	0,38	0,387	0,21	0,484	0,17	0,092	0,967
900	0,212	0,42	0,385	0,23	0,482	0,19	0,104	0,963
1000	0,211	0,47	0,384	0,26	0,479	0,21	0,116	0,959
1100	0,210	0,52	0,382	0,29	0,477	0,23	0,128	0,955
1200	0,209	0,57	0,380	0,32	0,475	0,25	0,140	0,950
1300	0,208	0,62	0,378	0,34	0,473	0,27	0,153	0,946
1400	0,207	0,68	0,377	0,37	0,471	0,30	0,165	0,941
1500	0,206	0,73	0,375	0,40	0,468	0,32	0,178	0,937
1600	0,205	0,78	0,373	0,43	0,466	0,34	0,191	0,932
1700	0,204	0,83	0,371	0,46	0,464	0,37	0,204	0,928
1800	0,203	0,89	0,369	0,49	0,462	0,39	0,217	0,923
1900	0,202	0,94	0,367	0,52	0,459	0,41	0,230	0,919
2000	0,201	0,99	0,366	0,55	0,457	0,44	0,243	0,914
2100	0,200	1,05	0,364	0,58	0,455	0,46	0,257	0,909
2200	0,199	1,11	0,362	0,61	0,452	0,49	0,270	0,904
2300	0,198	1,16	0,360	0,64	0,450	0,51	0,284	0,899
2400	0,197	1,22	0,358	0,67	0,447	0,54	0,298	0,894
2500	0,196	1,28	0,356	0,70	0,445	0,56	0,312	0,889
2600	0,195	1,34	0,354	0,74	0,442	0,59	0,327	0,884
2700	0,193	1,40	0,352	0,77	0,440	0,61	0,341	0,879
2800	0,192	1,46	0,350	0,80	0,437	0,64	0,356	0,874
2900	0,191	1,52	0,347	0,83	0,434	0,67	0,371	0,868
3000	0,190	1,58	0,345	0,87	0,432	0,70	0,386	0,863
3100	0,189	1,64	0,343	0,90	0,429	0,72	0,402	0,858
3200	0,187	1,71	0,341	0,94	0,426	0,75	0,417	0,852
3300	0,186	1,77	0,339	0,97	0,423	0,78	0,433	0,846
3400	0,185	1,84	0,336	1,01	0,420	0,81	0,449	0,841
3500	0,184	1,91	0,334	1,05	0,417	0,84	0,466	0,835
3600	0,182	1,97	0,332	1,09	0,414	0,87	0,483	0,829
3700	0,181	2,04	0,329	1,12	0,411	0,90	0,500	0,823
3800	0,180	2,11	0,327	1,16			0,517	0,817
3900	0,178	2,19	0,324	1,20			0,535	0,810
4000	0,177	2,26	0,322	1,24			0,553	0,804
4100	0,175	2,34					0,571	0,798
4200	0,174	2,41					0,590	0,791
4300	0,172	2,49					0,609	0,784
4400	0,171	2,57					0,629	0,777
4500	0,169	2,66					0,650	0,770
4600	0,168	2,74					0,670	0,762
4700	0,166	2,83					0,692	0,755

$$A_a = \frac{M_u}{k_a \cdot h} \text{ of } A_a = \omega_0 \cdot b \cdot h \cdot 10^4; \qquad x = k_x \cdot h; \qquad z = k_z \cdot h$$

M_u in kNm; b in m; h in m; A_a in mm^2; $\frac{M_u}{bh^2}$ in kN/m^2

FeB	$k_{x\,\max}$	ω_0	$0{,}6\,k_{x\,\max}$	ω_0
220	0,695	2,84	0,417	1,71
400	0,555	1,25	0,333	0,75
500	0,500	0,90	0,300	0,54

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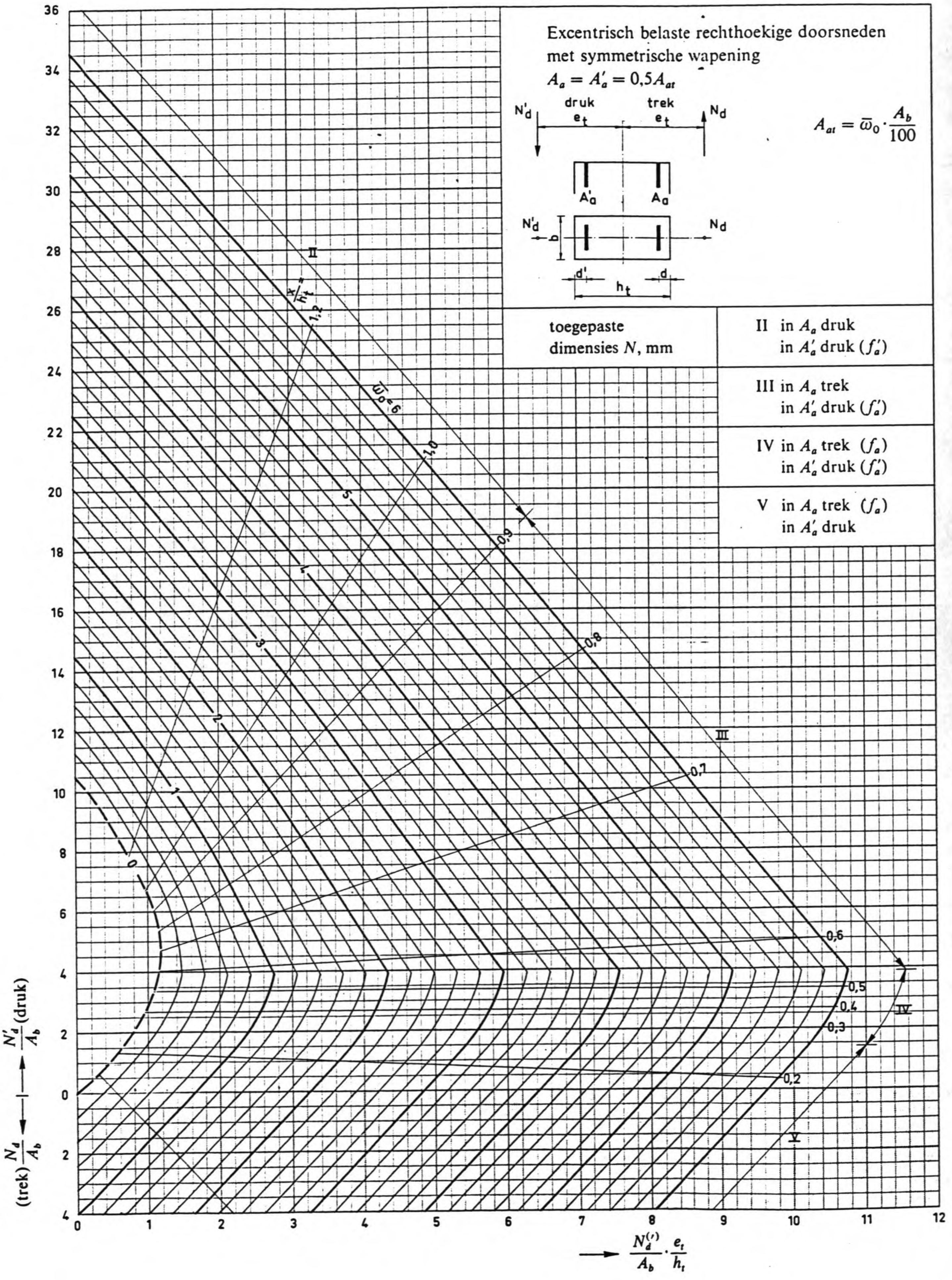
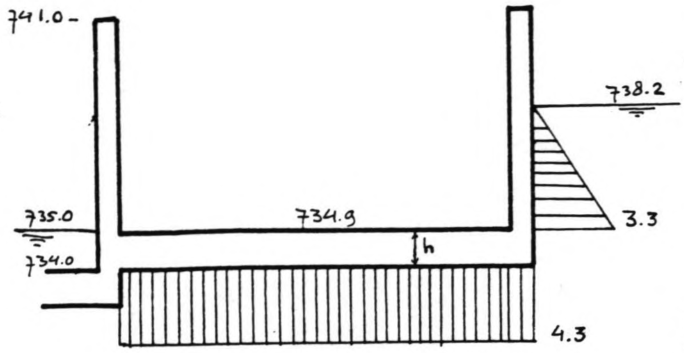


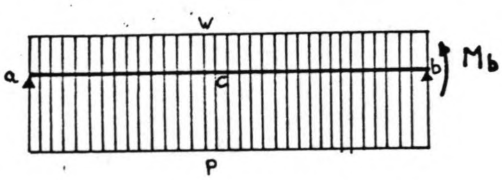
Doorsnede van betonstaalstaven in mm² per meter plaatbreedte

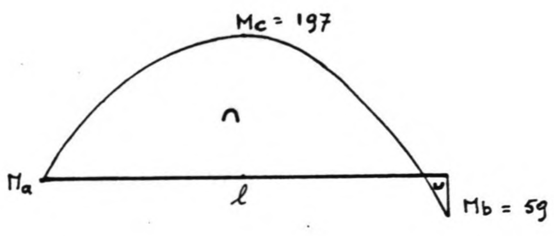
afstand h.o.h. in mm	aantal staven per m	middellijn $\varnothing_k$ in mm								
		6	8	10	12	16	20	25	32	40
70	14,29	404	718	1122	1616	2872	4488	7012	11489	17952
75	13,33	377	670	1047	1508	2681	4189	6545	10723	16830
80	12,50	353	628	982	1414	2513	3927	6136	10053	15708
85	11,76	333	591	924	1331	2365	3696	5775	9462	14836
90	11,11	314	559	873	1257	2234	3491	5454	8936	13963
95	10,53	298	529	827	1190	2116	3307	5167	8466	13265
100	10,00	283	503	785	1131	2011	3142	4909	8042	12566
105	9,52	269	479	748	1077	1915	2992	4675	7660	11995
110	9,09	257	457	714	1028	1828	2856	4462	7311	11424
115	8,70	246	437	683	983	1748	2732	4268	6993	10948
120	8,33	236	419	654	942	1676	2618	4091	6702	10472
125	8,00	226	402	628	905	1608	2513	3927	6434	10069
130	7,69	217	387	604	870	1547	2417	3776	6187	9666
135	7,41	209	372	582	838	1489	2327	3636	5957	9321
140	7,14	202	359	561	808	1436	2244	3506	5745	8976
145	6,90	195	347	542	780	1387	2167	3385	5547	8677
150	6,67	188	335	524	754	1340	2094	3272	5362	8378
155	6,45	182	324	507	730	1297	2027	3167	5189	8116
160	6,25	177	314	491	707	1257	1963	3068	5027	7854
165	6,06	171	305	476	685	1219	1904	2975	4874	7623
170	5,88	166	296	462	665	1183	1848	2887	4731	7392
175	5,71	162	287	449	646	1149	1795	2805	4596	7187
180	5,56	157	279	436	628	1117	1745	2727	4468	6981
185	5,41	153	272	425	611	1087	1689	2653	4347	6798
190	5,26	149	265	413	595	1058	1653	2584	4233	6614
195	5,13	145	258	403	580	1031	1611	2517	4124	6449
200	5,00	141	251	393	565	1005	1571	2454	4021	6283
205	4,88	138	245	383	552	981	1532	2395	3923	6134
210	4,76	135	239	374	539	957	1496	2337	3830	5984
215	4,65	132	234	365	526	935	1461	2283	3741	5848
220	4,55	129	228	357	514	914	1428	2231	3656	5712
225	4,44	126	223	349	503	894	1396	2182	3574	5588
230	4,35	123	219	341	492	874	1366	2134	3497	5464
235	4,26	120	214	334	481	856	1337	2089	3422	5350
240	4,17	118	209	327	471	838	1309	2045	3351	5236
245	4,08	115	205	321	462	821	1282	2004	3283	5137
250	4,00	113	201	314	452	804	1257	1963	3217	5037

E.2. Section I. Desilting basin

Maximum load:
- maximum design flood: TWL = 738.2 m.EL
- empty basin
- closed gates: sluiceway WL = TWL
  (for walls and floor ends)
- open gates: sluiceway water depth = 1.0 m







FLOOR (h = 1.0 m.
l = 10.0 m.)

weight $w = h * \rho_c * g = 24.0$ kN/m
hydraulic pressure
$p = \Delta H*g = 4.3*9.8 = 42.1$ kN/m

$M_b = 1/6 * 3.3^3 * g = 59$ kNm
$M_c = 1/8 * (p - w) * l^2 - M_b/2$
$= 197$ kNm

$\gamma * M_c / (b*h^2) = 1.7 * 197 / 1$
$= 334$ kN/m²

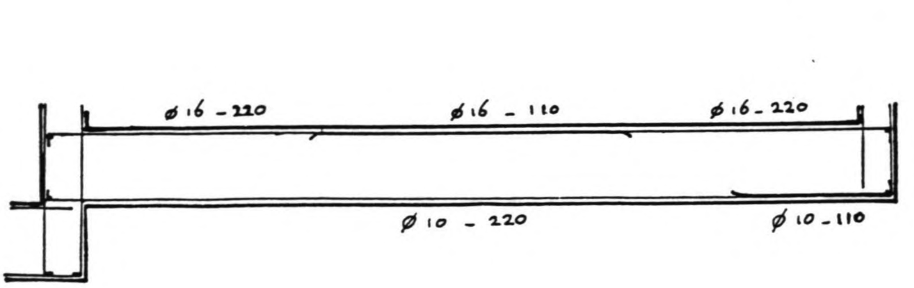
$\Rightarrow \omega_o \approx 0.16$ (table E.1)
$A_a = \omega_o * b * h * 10000$
$= 1600$ mm²/m

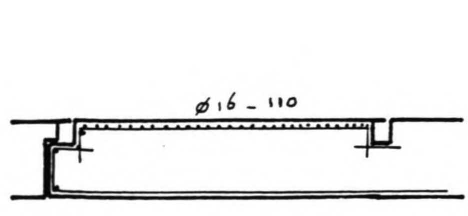
Ø 16 - 110 (= 1828 mm²/m; table E.2)

$\gamma * M_b / (b*h^2) = 100$ kN/m²

$\Rightarrow \omega_o = 0.05$
$A_a = 500$ mm²/m

Ø 10 - 110 (= 714 mm²/m)





Shear $S_b = 1/2 * l * (p - w) = 91$ kN

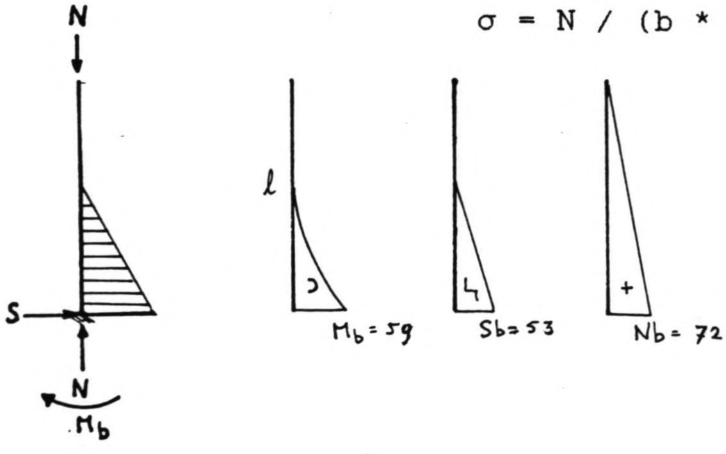
$\gamma * \tau = 1.7 * S_b / (b * h) = 0.14$ N/mm²

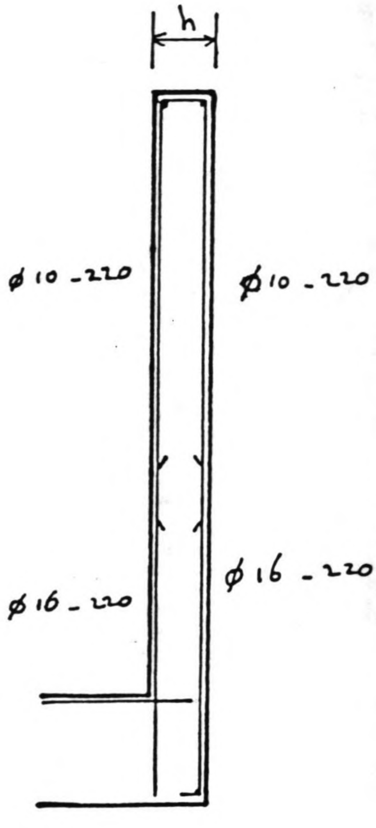
< 0.55 N/mm, so no shear reinforcement necessary.

WALLS (h = 0.5 m.
l = 6.1 m.)

weight N = l * h * $_c$ * g
= 6.1 * 0.5 * 2.4 * 9.8
= 72 kN

σ = N / (b * h) = 0.15 N/mm²





$M_a = M_b = 1/6 * H^3 * g$
$= 1/6 * 3.3^3 * 9.8$
= 59 kNm

e = γ * M / N = 1.4 m.
σ * e / h = 0.4 N/mm²

=> $\omega_o \approx$ 0.2 * (400/220) = 0.36
(figure E.2)

A_a = 1800 mm

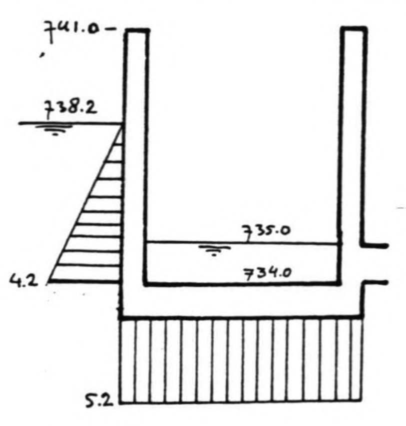
Ø 16 - 220 on each side

S_b = 1/2 * H² * g = 53 kN
* τ = 0.18 N/mm² (< 0.55)

==

E.3. Section I. Sluiceway

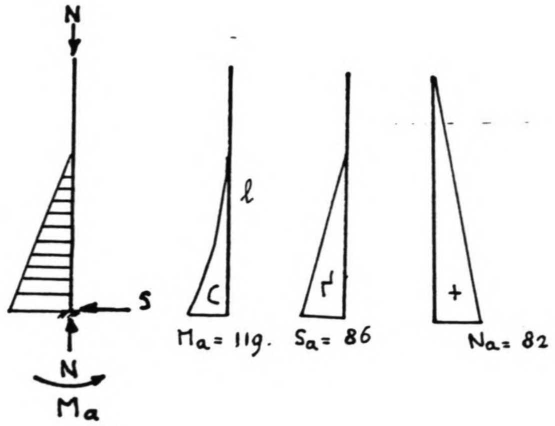
Maximum load: - maximum flood level: TWL = 738.2 m.EL
- empty desilting basin
- open gates



OUTER WALL (h = 0.5 m.
l = 7.0 m.)

weight N = 7.0 * 0.5 * 2.4 * 9.8
= 82 kN

σ = 82 / 500 = 0.16 N/mm²



$M_a = 1/6 * (4.2^3 - 1.0^3) * 9.8$
$= 119$ kNm

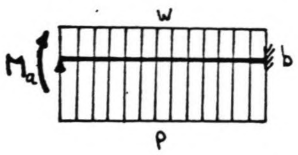
$e = 1.7 * 119 / 82 = 2.5$ m.
$\sigma * e / h = 0.8$ N/mm²

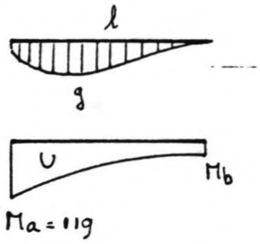
$=> \omega_o \approx 0.40$
$A_a = 2000$ mm²/m

∅ 16 - 200 (on each side)

$S_a = 1/2 * (4.2^2 - 1.0^2) * 9.8$
$= 86$ kN

$\gamma * \tau = 0.29$ N/mm² (< 0.55)





FLOOR (h = 1.0 m.
l = 5.0 m.)

weight w = 24.0 kN/m
press. p = (5.2 - 1.0) * 9.8
= 41.2 kN/m²

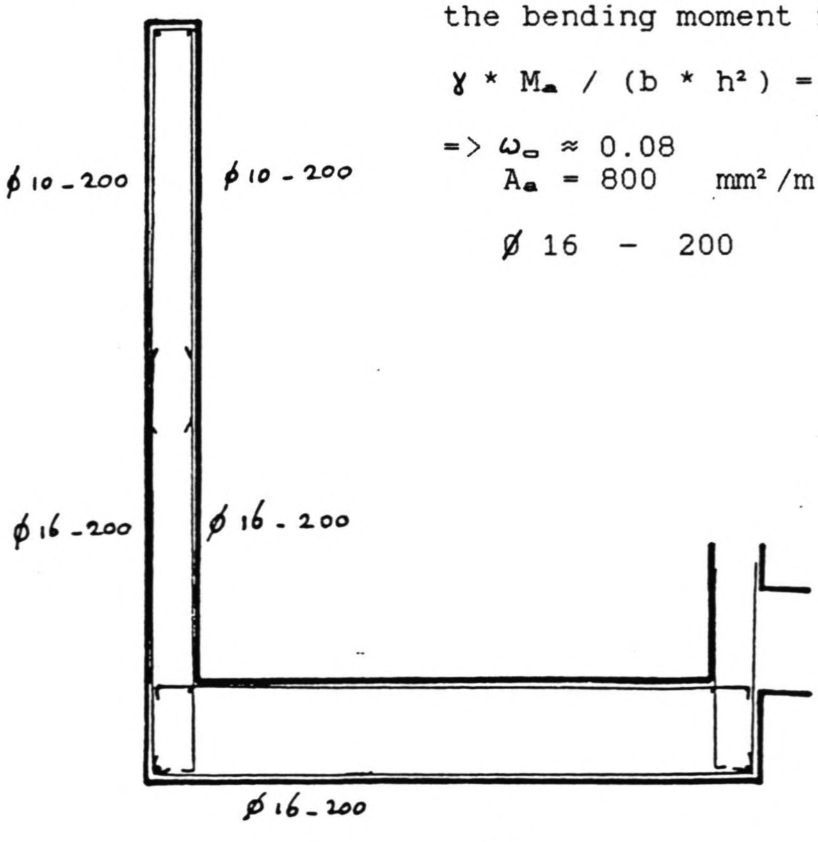
$M_a = 119$ kNm
$M_b = M_a - 1/8 * (p - w) * l^2$
$= 119 - 20 = 99$ kNm

This means there is no positive bending moment in the floor, so there will be a ground pressure along the whole floor that reduces the bending moment in b.

$\gamma * M_a / (b * h^2) = 168$ kN/m²

$=> \omega_o \approx 0.08$
$A_a = 800$ mm²/m

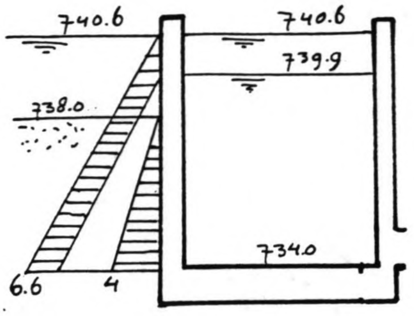
∅ 16 - 200

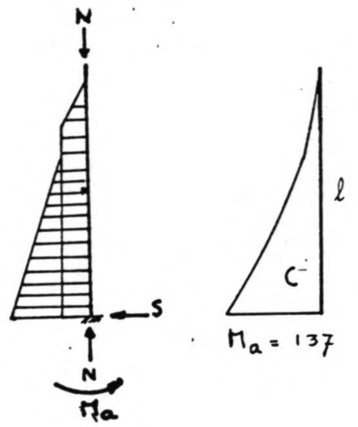


==

E.4. <u>Section II. Sluiceway walls</u>

Maximum load: - maximum flood level: BWL = 740.6 m.EL
- gates open (for outer wall):
sluiceway WL = 739.9 m.EL
- gates closed (for inner wall):
sluiceway WL = BWL





OUTER WALL (h = 0.5 m.
l = 7.0 m.)

weight N = 7.0 * 0.5 * 2.4 * 9.8
= 82 kN

σ = 82 / 500 = 0.16 N/mm²

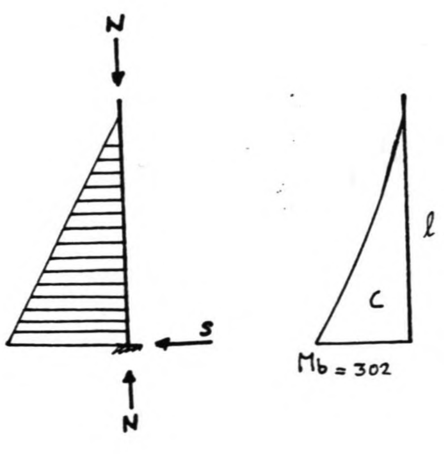
ground pressure:
$p_g = h * \rho_g * \tan(45° - Ø/2) * g$
$= h * 1.25 * \tan 30° * 9.8$
$\doteq 4.1 * h$

$M_a = 1/6 * (6.6^3 - 5.9^3) * 9.8 +$
$+ 1/6 * 4.1 * 4$
$= 137$ kNm

e = 1.7 * 137 / 82 = 2.8 m.
σ * e / h = 0.9 N/mm²

=> ω_o = 0.5
A_a = 2500 mm²

Ø 16 - 150 on each side



INNER WALL (h = 0.5 m.
l = 6.1 m.)

weight N = 72 kN
= 0.14 N/mm²

$M_b = 1/6 * 5.7^3 * 9.8 = 302$ kNm

e = 1.7 * 302 / 72 = 7.1 m.
σ * e / h = 2.0 N/mm.

=> $\omega_o \approx 1.1$
A_a = 5500 mm²

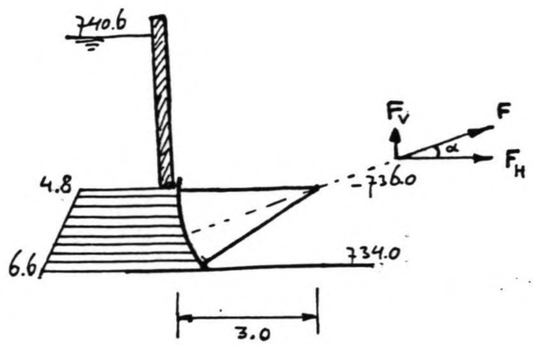
Ø 16 - 75 on each side

$S_b = 1/2 * 5.7^2 * 9.8 = 159$ kN
$\gamma * \tau = 1.7 * 0.318$
$= 0.54$ N/mm² (< 0.55)

==

E.5. Section III. Gate supports and dividing wall

Maximum load: - maximum flood level: BWL = 740.6 m.EL
- gates closed



$F = 1/2 * A * p_{avg}$
$= 1/2 * (1.8 * 2.1) * 56$
$= 106$ kN

$F_H = F * \cos\alpha$
$= 106 * 0.96 = 101$ kN
$F_V = F * \sin\alpha = 30$ kN

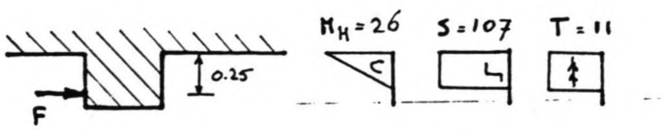
SUPPORTS (b = 0.5 m, h = 0.3 m)

$M_H = F_H * 0.25 = 25$ kNm

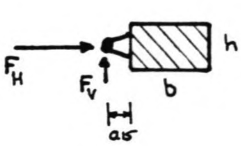
$\gamma * M_H / (h * b^2) = 666$ kN/m²

$\Rightarrow \omega_o = 0.31$
$A_a = 465$ mm²

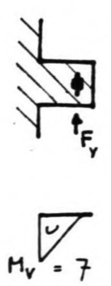
3 * Ø 16



top view



side view



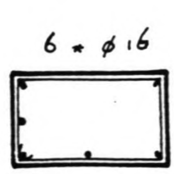
front view

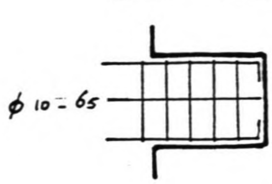
$M_V = F_V * 0.25 = 8$ kNm

$\gamma * M_V / (b * h^2) = 330$ kN/m²

$\Rightarrow \omega_o = 0.15$
$A_a = 225$ mm²

2 * Ø 16



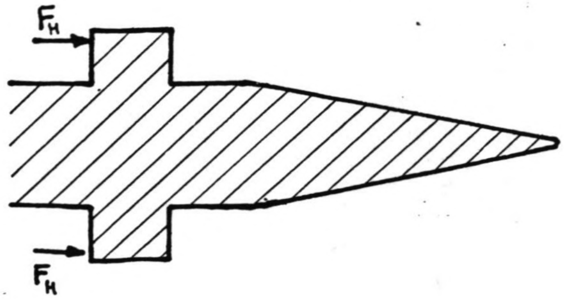


$S = F_H * (1 / \cos\alpha) = 102$ kN
$T = F_V * (0.15 + b/2) = 13$ kNm

$\tau = 0.85 * \{S/(b*h) + \Psi * T/(h*b^2)\}$
$= 0.85 * \{0.68 + 4.2 * 0.17\}$
$= 1.18$ N/mm²

$A_a = (\gamma * 1.18 - 0.55)*h*1000/220$
$= 2480$ mm²

$\Rightarrow$ Ø 10 - 65



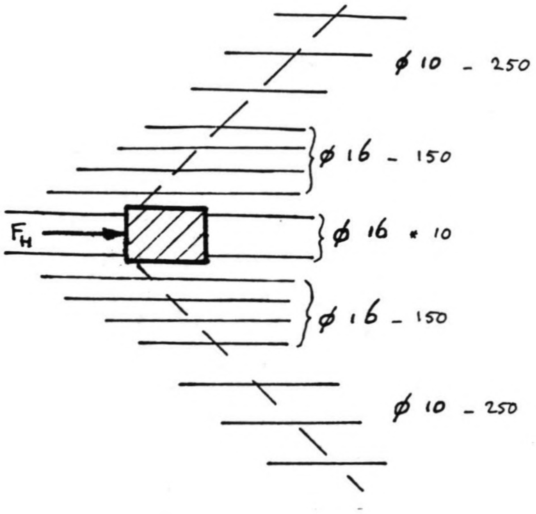
WALL (b = 0.8 m.)

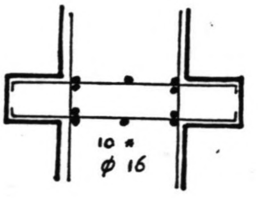
$N = 2 * F_H = 212$ kN

$\gamma * \sigma = 2 * 212*1000/300*800)$
$= 1.8$ N/mm²
> 1.1 N/mm², so reinforcement is necessary.

$A_a = \gamma * N / 220$
$= 2 * 212 / 220 = 1927$ mm

$\Rightarrow 10 * \varnothing 16$





decreasing reinforcement

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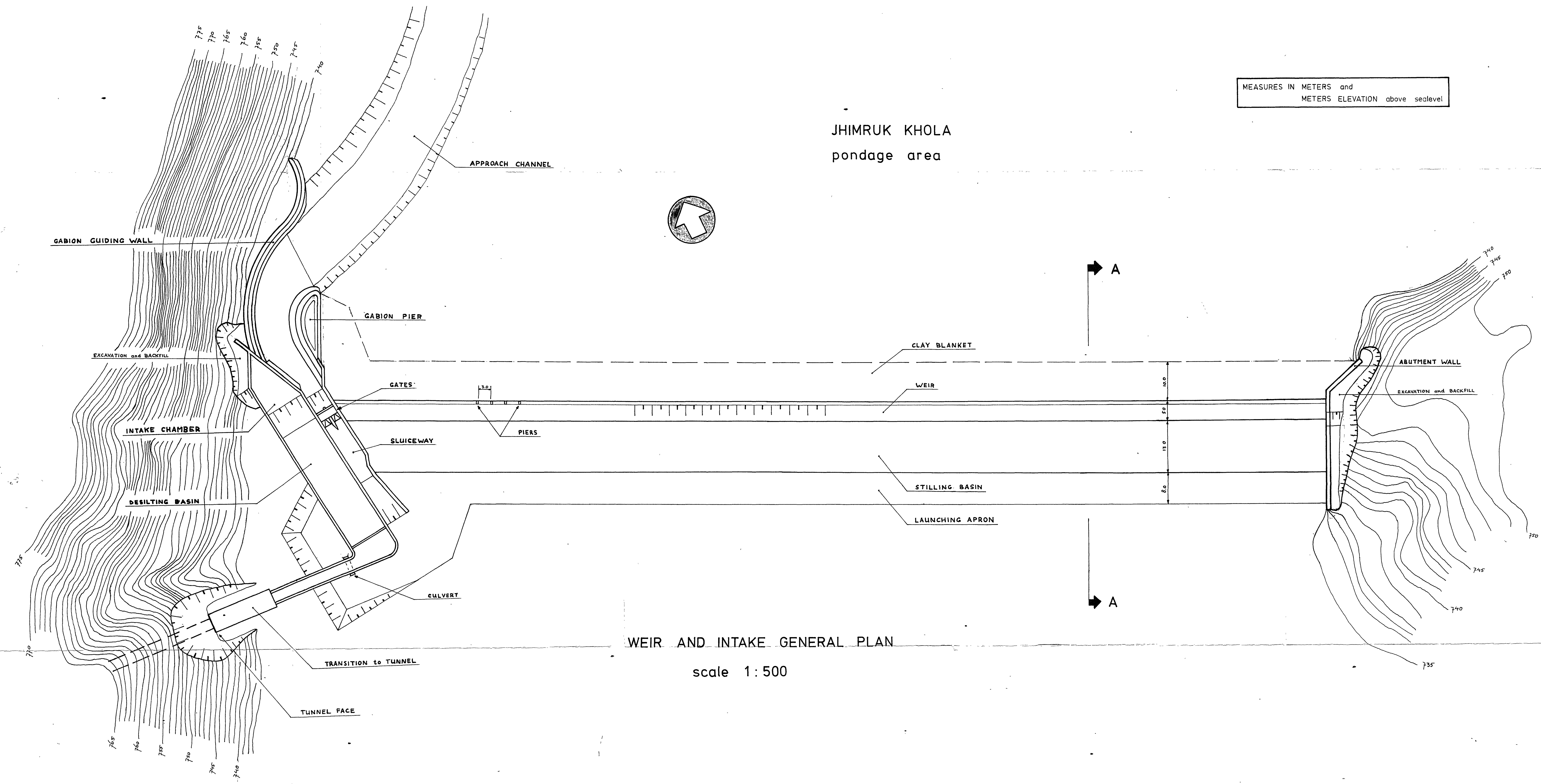
Third Edition


TUNNEL CROSS SECTION
scale 1 : 50
PCC
R = 1.25
masonry
concrete
0.50
2.50
0.50
3.50
1.25
1.25
MEASURES IN METERS and
METERS ELEVATION above sealevel
JHIMRUK KHOLA
MARI KHOLA
800
363
PROJECT GENERAL LAYOUT scale 1 : 2000
steel 6 - 12 mm
concrete
steel clasp
1.00
1.00
2.50
1.50
SURFACE PENSTOCK
SUPPORT
CROSS SECTIONS
scale 1 : 50
INTAKE
TUNNEL
1 : 1000
EL 736.1
SURGE TANK
EL 735.3
42
800
PENSTOCK
underground penstock alternative
POWER HOUSE
EL 551.5
EL 545.0
EL 520.0
180
183
LONGITUDINAL SECTION
scale 1 : 2000
steel 6 - 12 mm
concrete
R = 0.35
UNDERGROUND PENSTOCK SECTION
scale 1 : 50
JHIMRUK
HYDRO-ELECTRIC PROJECT
DRAWING NO. 1
General layout
Longitudinal section
Tunnel & penstock sections
ASSIGNMENT BY : BPC Hydroconsult
Kathmandu, Nepal
GENERAL DESIGN : NEPAL ELECTRICITY AUTHORITY
DRAWN BY : P J. PLOOY
PLACE : DELFT , HOLLAND
DATE : 16 - 8 - 1988

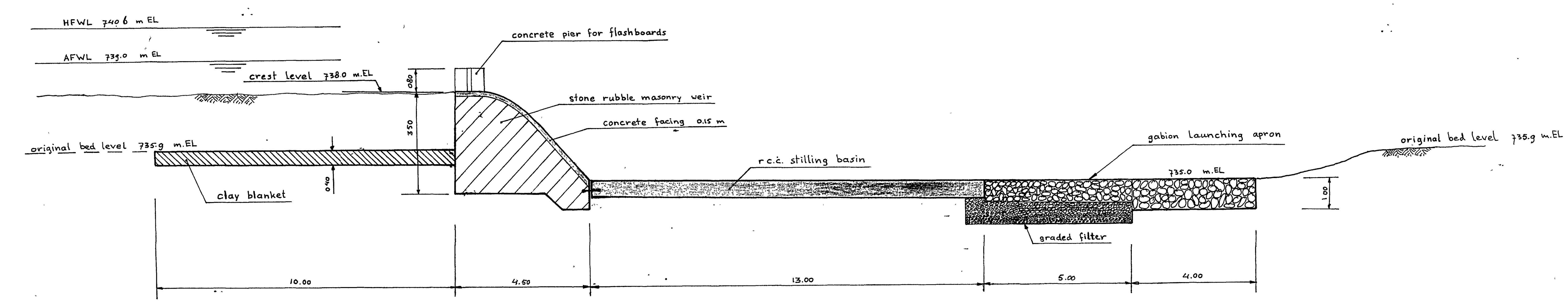
DRAWING #1 GENERAL LAY OUT
LONGITUDINAL SECTION
TUNNEL AND PENSTOCK SECTIONS

MEASURES IN METERS and
METERS ELEVATION above sealevel



WEIR AND INTAKE GENERAL PLAN

scale 1 : 500



WEIR SECTION A - A

scale 1 : 100

WATER LEVELS:
AFWL = Annual Flood Water Level
HFWL = High Flood Water Level

JHIMRUK KHOLA
HYDRO-ELECTRIC PROJECT

DRAWING NO. 2

Weir and intake plan
Weir cross section

ASSIGNMENT BY.
BPC Hydroconsult
Kathmandu, Nepal

DESIGN BY: P. J. PLOOY

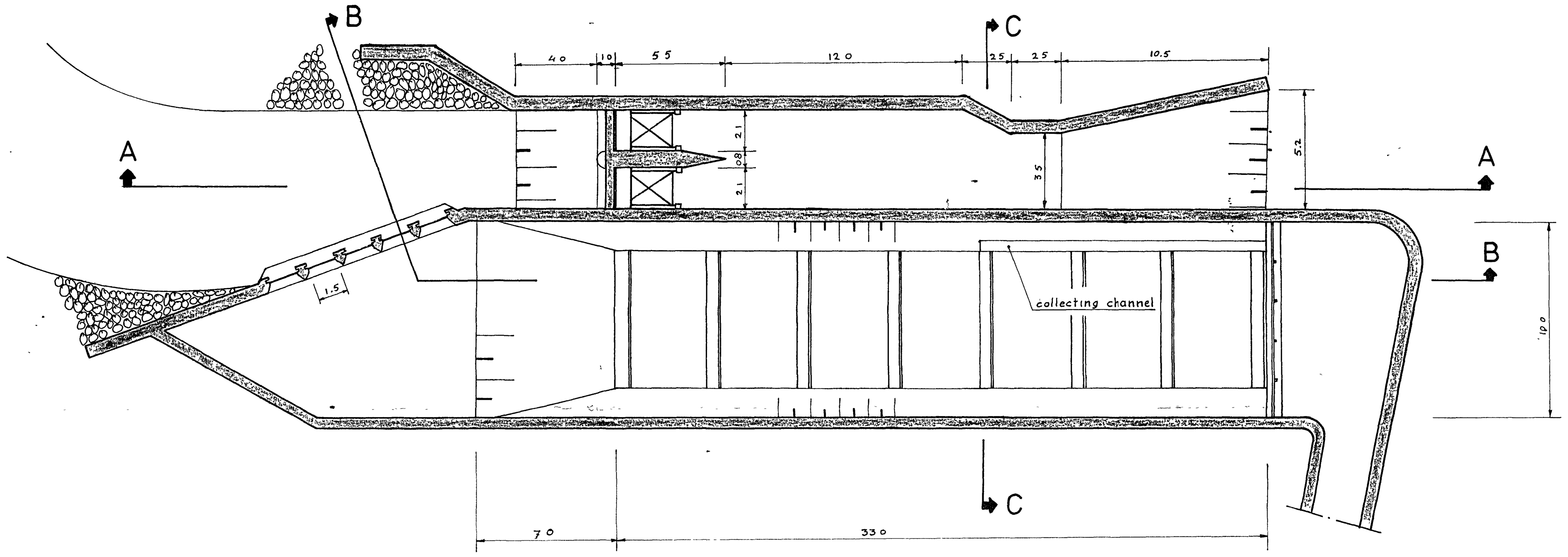
DRAWN BY: P.J. PLOOY

PLACE: DELFT, THE NETHERLANDS

DATE: 31 - 8 - 1988

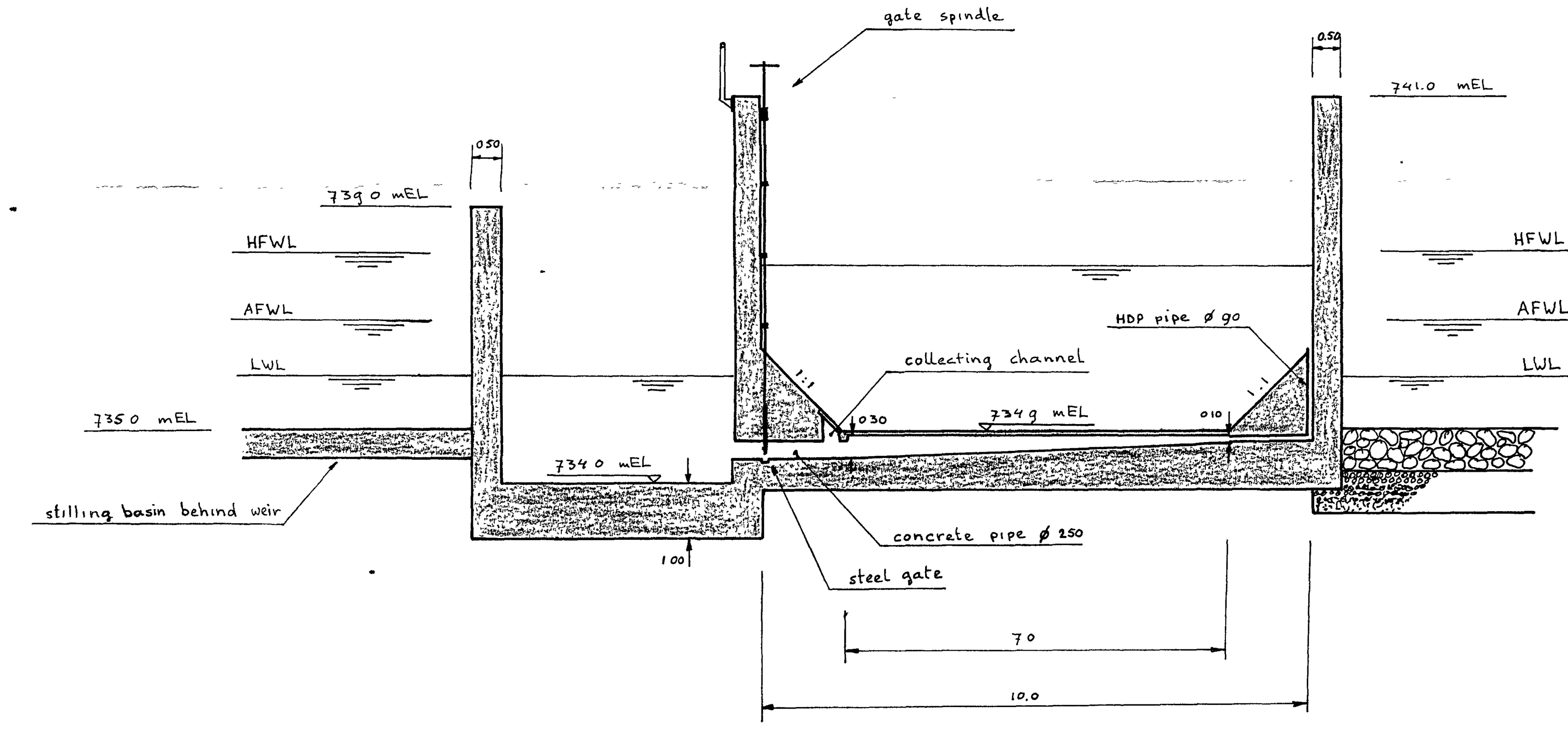
DRAWING #2 WEIR AND INTAKE PLAN
WEIR CROSS SECTION

MEASURES IN METERS and
METERS ELEVATION above sealevel



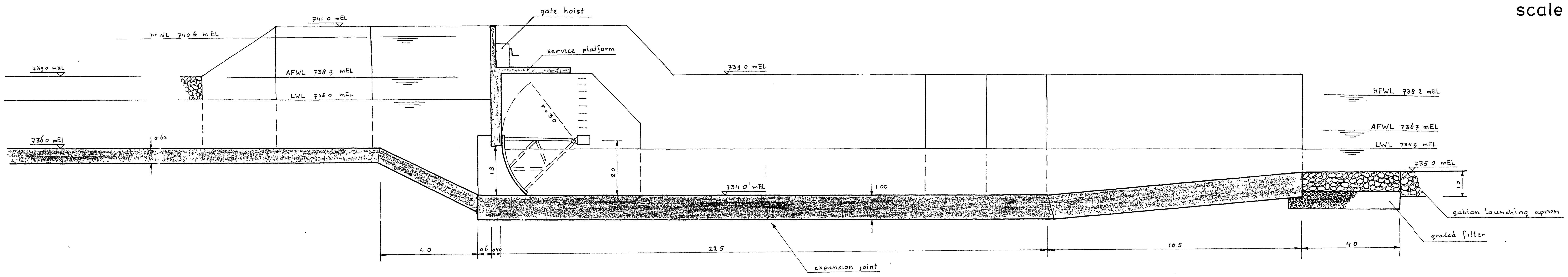
DESILTING BASIN and SLUICEWAY PLAN

scale 1 : 200



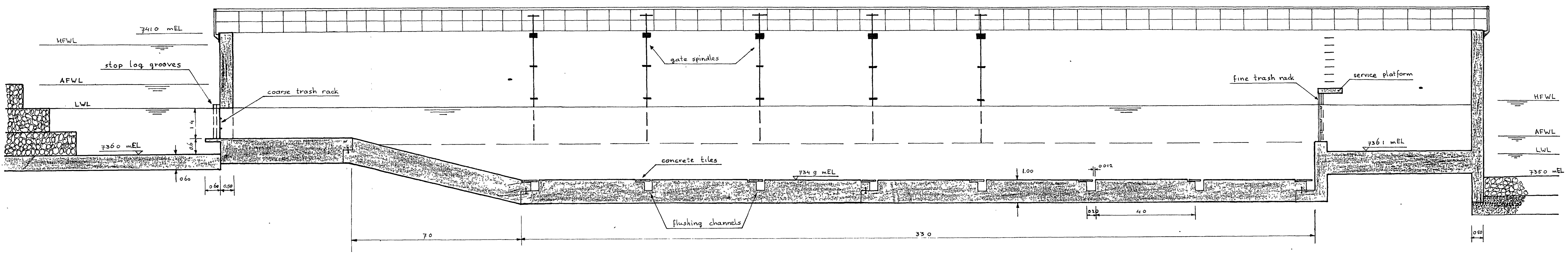
FLUSHING SYSTEM CROSS SECTION C - C

scale 1 : 100



SLUICEWAY SECTION A - A scale 1 : 100

WATER LEVELS :
HFWL = High Flood Water Level
AFWL = Annual Flood Water Level
LWL = Low Water Level



DESILTING BASIN SECTION B - B scale 1 : 100

JHIMRUK
HYDRO - ELECTRIC PROJECT

DRAWING NO 3
Desilting basin and sluiceway sections

ASSIGNMENT BY.
BPC Hydroconsult
Kathmandu , Nepal

DESIGN BY: P. J. PLOOY

DRAWN BY: P J PLOOY

PLACE DELFT , THE NETHERLANDS

DATE : 27 - 10 - 1988

DRAWING #3 DESILTING BASIN AND SLUICEWAY SECTIONS

883619