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REPORT ON STANDARDIZATION OF CIVIL WORKS FOR SMALL HYDROPOWER PLANTS*

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CHAPTER 1

1. INTRODUCTION

The UNIDO acting as a centralized agency, aims to collect and integrate in a simplified form data and information relating to small hydroelectric plants, thus enabling developing countries to provide electric power in remote or isolated areas.

Recognizing that the main problems impending the development of such plants are on the one hand the lack of simple generally applicable procedures and methods of realization and on the other hand , the disproportion between design costs and total cost of the installed plants, a serious effort has been undertaken to standardize the design and the main components of such plants .

Whereas the standardization of the mechanical and electrical components is relatively easy, standard designs for the civil construction have been more difficult to pinpoint.

The present paper aims at filling at least partly this gap, by providing

- designs simple to realize
- simplified data and practical calculation methods.

It is evident that this kind of efforts at standardization can only be based on not inconsiderable experience - to its design, realization and operation - obtained in countries in which hydroelectric power generation has traditionally been utilized over periods of many years.

Standardization of small hydroelectric plants for countries in which such plants are required brings about a number of advantages, the most important ones being:

◇ easier and simplified realization

◇ minimum design costs

◇ enabling local, possibly less highly trained personnel, to take in hand their realization

◇ maximazing the use of locally obtained components (civil, mechanical and electrical)

◇ enabling technological transfer

◇ reducing expediture of foreign currency

◇ encouraging local manufacture and contracting

It is evident that the present paper does not pretend to solve the question of standardization once and for all; the long-term objective of UNIDO would however be to provide all countries - particularly developing countries - in need of small hydro plants, with enough "tools" to initiate and pursue their own development. The present standardization effort in sections of components of a "ready made" form, as summarized in this paper with the addition of worked-out examples (drawings and specifications) should represent a significant first step in fulfilling the above objective.

It will be noted that our Swedisch colleagues have concentrated their efforts on the standardization of electromechanical equipment, whereas as already mentioned above our, that is to say, Electrowatt Engineering Service's efforts have been directed mainly towards the Civil Engineering aspects of small and mini hydropower stations.

CHAPTER 2

2. THE PHILOSOPHY OF STANDARDIZATION OF SMALL HYDROPOWER SCHEMES

There are practically no two hydropower schemes that can be duplicated.

This notwithstanding, the aim of the present study is to provide general guidance in regard to the design and realization of the most important components of such plants; in other words, to provide basis of calculation, sketches and drawings, which will be easy to interpret and adapt, even if the basic data of the actual plant to be constructed are somewhat different.

In attempting to deal with this type of approach, we have divided a regular normal hydropower scheme into its most useful components, and have endeavoured to develop each one of those components into independent items, which should within certain reasonable limits permit the unification of those parts, matching them together in what we for simplicity have opted to call the 'Lego System' (after the toys of the same name that small children in Switzerland use to construct the most varied structures with only a few basic elements).

By following this philosophy of a 'Lego System', and putting together different technological brickstones to form a unified scheme, we believe that it should be possible to achieve over the long term, a transfer of this type of elusive technology to less developed countries, which in turn should allow them to develop their own hydropower plants, using local materials and local human resources.

Before any design can be started, some data have to be gathered, being the most usual necessary data for the design and construction of micro, mini and small hydropower schemes; they are the following :

- ◇ Topographic data
- ◇ Geologic and geotechnical data
- ◇ Hydrological data

The hydrological data has to be carefully prepared in order to determine two of the main design parameters, i. e.: the optimal water discharge or Q_{design}, and the maximum water discharge $Q_{max.}$ which the structures will have to withstand during their lifetime.

Q_{design} is to be used in the dimensioning of the main structures such as the water intakes, the waterways, and the generating equipment. Q_{max} will warrant a safe design of the intake structure and provide security to people living downstream.

Armed with the above-mentioned information, a first rough design can be evolved. For this design, the above mentioned Lego System could be subdivided into the following independent parts:

- ◇ Water intake
- ◇ Desilting works, if required
- ◇ Waterways
- ◇ Forebay, if required
- ◇ Penstock
- ◇ Powerhouse
- ◇ Tailrace.

Each one of these parts will have to fullfill certain natural and technical conditions as we will see further on.

The first "Lego" stone "The Water Intake" will deal with two different types of water intakes, the first one covering intakes for the runoff river type, the second one being for the tyroler or drop types.

Whereas the following Table No. 2.1 indicates the most usual design discharges applying to small hydroelectric plants, it will be noted that details of water intakes are given for three different discharges for run-off-river plants and one for the tyroler type.

These details should enable the design engineers a certain amount of interpolation should their own data not coincide with those indicated in the table.

Alternatively, new data and drawings could be established by extrapolation, if the above interpolation is insufficient.

TABLE 2.1
MATRIX OF WATER INTAKES EXAMPLES

TYPE OF INTAKE	DESIGN DISCHARGE (m^3/s)						
	0.5	1.0	1.5	2.0	2.5	3.0	3.5
RUN OFF RIVER		SEE EXAMPLE No. 1			SEE EXAMPLE No 2		SEE EXAMPLE No 3
DROP OR TIROLER		SEE EXAMPLE No. 4					

The second "Lego" stone refers to the "Desilting Structures", which in this report, include only the most simple types available which are particularly suitable for use in developing countries.

The authors have refrained from including in this report more sophisticated desilting systems, because many of their components would have to be imported. Local manufacture would be impossible, and therefore the cost of the structures would become high, and foreign exchange expediture would be unavoidable. We have listed these simple standardized desilting structures in the following table:

TABLE 2.2
MATRIX OF DESILTING EXAMPLES

TYPE OF DESILTING	DESIGN DISCHARGE (m^3/s)						
	≤1.0	1.5	2.0	2.5	3.0	3.5	5.0
RUN OFF RIVER	Included in the Intake			SEE EXAMPLE No. 2		SEE EXAMPLE No 3	
DROP OR TIROLER	SEE EXAMPLE No. 4						

The third "Lego" stone is the the adduction way or canal. It is pertinent to recall here that the design parameters of a canal can have an infinite number of combinations, however, some examples will help the designer of specific plants to take decisions and to properly design their canals.

The adduction way may sometimes include tunnels. However, the authors have avoided including them in this report, in order to concentrate on other items considered to be more important. In any case, the decision to be taken if a tunnel should or should not be used, remains mostly an economic decision.

Should the user of this manual require more specific knowledge in the field of canals, we recommend to consult the excelent book issued by the Bureau of Reclamation of the United States of America, entitled "Design of Small Canal Structures".

The following table which could be completed later refers to three examples namely for Design Discharges of 1.0, 2.5 and 3.5 m^3/s

TABLE 2.3
MATRIX OF ADDUCTION WAYS AND CANALS EXAMPLES

TYPE OF CANAL	DESIGN DISCHARGE (m^3/s)						
	1.0	1.5	2.0	2.5	3.0	3.5	5 0
TRAPEZOIDAL	SEE FIGURE 8-2			SEE FIGURE 8-2		SEE FIGURE 8-2	

The fourth "Lego" stone is the Forebay, for which three examples are given as a function of design discharge and regulation volumes :

TABLE 2.4
MATRIX OF FOREBAYS EXAMPLES

	DESIGN DISCHARGE (m^3/s)						
	1.0	1.5	2.0	2.5	3.0	4.0	5 0
FOREBAY	SEE EXAMPLE No 5			SEE EXAMPLE No 6			

The fifth "Lego" stone is the penstock structure, which is difficult to standardize, because no two penstocks are alike; They have to be tailored to the available conditions.

The following Table, which could be completed at a later date indicates an example for heads of 100-200 m and a design discharge of 1.0 and 2.5 m^3/s.

TABLE 2.5
MATRIX OF PENSTOCKS EXAMPLES

HEAD (m)	DESIGN DISCHARGE (m^3/s)						
	1.0	1.5	2.0	2.5	3.0	4.0	5.0
5 - 10							
10 - 20							
20 - 50							
50 - 100	SEE EXAMPLE No. 7			SEE EXAMPLE No 8			
100 - 200							
> 200							

The sixth "Lego" stone is the powerhouse of which two aspects have to be considered. The first concernsthe determination of the type and the sizes of the machines to be used. A second relates mainly to the Civil Works. The electromechanical part, equipment etc. has not been treated in this report. It can be found in other previous reports written for this purpose; the "Report on standardization of small hydropower plants ", specially made for the UNIDO by the Swedish Consulting firm SWECO, deals mainly with the first aspect.

The relevant sixth "Lego" stone matrix includes examples worked out for Tailraces valid for various types of discharges, heads, and types of turbines.

TABLE 2.6
MATRIX OF POWERHOUSE EXAMPLES

TYPE OF TURBINE	DESIGN DISCHARGE (m^3/s)						
	1.0	1.5	2.0	2.5	3.0	4 0	5.0
CROSS FLOW							
FRANCIS				SEE EXAMPLE No. 9			
PELTON							

Now that the proposed philosophy of this report is evident, the following chapters of this report, will present the design criteria employed for small schemes with some worked out examples as a guide.

The Tailrace can be handled like the third Lego stone.

Important items like the preparation of concrete, the calculation and disposition of reinforced steel, etc., have not being covered in this report, as well as certain parts of hydroprojects that are less common or not economical in small schemes, like tunnels and surge chambers, which have also not being included.

Chapter 3

3. TOPOGRAPHY

Mapping and surveying belong to the basic activities of indisputable importance for any project in civil engineering. In isolated areas of overseas regions usually no adequate maps exist and special maps have to be established on suitable scales. The work comprises especially the following items:

- ◇ Procurement of aerial photographs (if available)
- ◇ Establishment of basic horizontal and vertical controls
- ◇ Topography survey
- ◇ Preparation of topographic maps
- ◇ Survey returns

3.1 Establishment of basic horizontal and vertical controls

In general, all horizontal and vertical controls shall be tied to points of known geographic positions and elevations established by the relevant National Geodetic Survey Authorities.

The plane coordinate system is usually similar to the internationally adopted Universal Transverse Mercator Grid System (x-axis on the equator, y-axis on a full degree of longitude)

Different tidal datum planes are determined from continuous tidal observation series:

◇ Mean Higher High Water	(MHHW)
◇ Mean High Water	(MHW)
◇ Mean Tide Level	(MTL)
◇ Mean Sea Level	(MSL)
◇ Mean Low Water	(MLW)
◇ Mean Lower Low Water	(MLLW)

For small or very small isolated projects, some of the above-mentioned tidal datum planes can be omitted, depending on the importance of the project.

Usually the datum plane selected as reference is the Mean Sea Level (MSL).

The survey markers should be established with such a density as to provide the tacheometric survey, ensuring that continual protection to these points is guaranteed. Benchmarks should be brass rods set in concrete foundations, protruding about 4 mm from its surface.

Detailed descriptions of each survey mark must be made for future reference and recovery.

3.2 Topographic survey

The field measurements carried out by a surveyor fully familiar with the necessary measuring techniques should provide the following for the:

- Feasibility Study

  - General maps, scale 1 : 20 000 up to 1 : 5 000 with contour lines of 10 or 5 metres

- Detail Design

  - Detailed topographic maps, scale 1 : 2 000 up to 1 : 500 with contour lines of 2, 1 and 0.5 metres

The ground controls for the plan and height of the survey markers would have to be carried out using conventional or electronic distance measuring equipment and theodolite.

Levelling shall be made as forward and backward running between fixed elevation of basic survey or loop closure on the same bench mark.

Applying tachymetric method, the density of measurement terrain points should be:

- For feasibility study: maps at least 16 points per ha.

- For detail design: maps at least 36 points per ha.

3.3 Accuracy of maps

The required accuracy of the maps should be:

- For feasibility study maps
  ± 3 metres in position and
  ± 2 metres in elevation

- For detail design maps
  ± 0.3 metre in position and
  ±0.2 metre in elevation.

The expected accuracy of contour lines shall be delineated to represent the true elevation and shape of the ground. The following table indicates practical data for the useful contour lines for different terrain slopes (detail design).

TABLE 3.1
ACCURACY OF CONTOUR LINES

AREAS WITH SLOPE	USEFUL CONTOUR LINE
< 1 %	0.5 m
1 % - 5 %	0.5 - 1.0 m
> 5 %	1.0 m

All planimetric features which are well defined on the ground shall be plotted, so that the position on the finished maps is accurate to within 0.5 mm to the true coordinate position.

Spot elevations placed in the map shall have an accuracy of at least 1/3 of the basic contour interval.

Eighty-five percent (85%) of all elevations interpolated from the map's contour lines shall be correct within half of the contour interval. Not more than 5% shall show errors in excess of the contour interval.

Any contour line which can be brought within the above-mentioned vertical tolerance, by moving its plotted position by 0.5 mm in any direction, shall be considered acceptable. If, for any part of the area the ground is obscured by vegetation , other obstacles or details, the Surveyor will show contours by broken lines and in those areas additional tolerance will be permitted in regard to the accuracy of the contours.

3.4 Preparation of Topographic Maps

Coordinates and elevations shall be computed as results from field notes. This includes computations on traverses, triangulations, tide observation (if no bench mark is available), level network, astronomic observations and other computations.

A topographic map shall be plotted in appropriate scale indicating all features of terrain (contour lines), including roads, houses, structures, rivers, canals, etc., in accordance with internationally accepted standard practice.

The map shall contain all planimetric features which are visible or identifiable or interpretable from the ground including land use features, such as trails, boundaries of wooded areas, monumented controls, orchards, buildings, roads, municipalities, cities and other work of man, etc.

Elevation of saddle tops, roads, intersections, low points in depressions, lakes and ponds, if these exist shall be shown to the nearest tenth (0.10) of a meter.

All planimetric and topographic features appearing in two adjoining map sheets shall match along the common projection line.

Coordinate grid designation shall be shown on the map at all sides of the neat line of the sheets.

All horizontal and vertical control points located within or near the map area shall be shown and designated by appropriate symbols, number and elevation, wherever applicable.

3.5 Information on Topographic Maps

All maps sheets shall show the following marginal information:

- North arrow
- Sheet index showing important details as lakes, shorelines mark towns and/or cities
- Legend of map and scale bar
- Contour interval
- Projection system used
- Geodetic datum used
- Government or private entities who prepared the maps
- Neat line of map sheet limits

3.6 Survey Returns

The surveyor in charge, should provide the following information :

- Field notes and computations on traverses, triangulations, level network and other computations in the establishement of horizontal and vertical ground controls.
- Topographic maps on deformation-proof synthetic paper
- Horizontal and vertical control point descriptions on mylard sheets.

CHAPTER 4

4. GEOLOGY

It is of capital importance to determine at an early stage of the studies the geological conditions of the area in which it is intended to develop a hydroelectric power project, and to determine the geological details in the immediate neighborhood of the main structures of the project, details that will be used later as input data in the design phase as well as during the construction period of the project.

The geologic appraisal should consider at least the following aspects:

4.1 General description of the project area

The natural features of the area are reproduced on the topographic maps. The general practice is to superimpose the geological conditions upon the same topographic maps; this includes the relevant morphologiy and when necessary a hydrogeological resume.

4.2 Relation of existing documents

At the beginning of the design stage it is of good practice to compile a complete list of all existing documents i. e. :

- Available topographical data
- Available geological data from previous studies
- Previous engineering reports
- Existing data of other engineering studies
- Reports and maps of other studies in the region that do not affect directly the project but which could provide usefull technical clues.
- Available aerial photography

4.3 Geological data required for the design

For preparing the necessary data for the design of the main structures some geological information is required. The more usual geological data required are the following:

- Determination of the stability and resistance of the soil.
- Determination of the soil density condition.
- Distribution and condition pertaining to the phreatic water.
- Consideration of possible problems to be encountered related with the construction of underground works, if any (tunnels, galleries, etc.).
- Listing and defining all problems related with surface works.
- Assurance that construction materials and quarry areas with adequate quality materials are available.
- Examination of any possible seismic activity within the zone of the project, and if such conditions exist, establishing the expected magnitude and acceleration data.
- Investigation whether a boring campain is necessary or not.

4.4 Determination of the geological conditions of non-consolidated soils and rocks

It will be necessary to pin-point and describe the type and the thickness of unstable slopes, alluvial soils and to select the zones in which the future project can be constructed without undue risks. Once this is done, the kind and importance of corrective measures to be taken can be established and their technical and economical consequences evaluated. Such measures include for instance the design of retention structures, the placing of gabions, etc.

If possible, the potential dangers of avalanches or extreme floods and their possible control and timely prevention measures should be predicted and incorporated in the design.

4.5 Geological maps

The scale of the geological maps usually depends on the available cartographic material. The most commonly recommended scales are .

- For regional geologic - tectonic maps, depending on the size of the zone, scales of 1 : 50 000 to 1 : 100 000.

- Scales of 1 : 5 000 to 1 : 10 000 , and in the case of unavailability of those scales, the scale of 1 : 25 000 are used for detailed geological maps for small dams, lay out of channels etc., .
- Geologic maps pertaining to important local zones of construction, such as areas for the water intake, the powerhouse, quarry areas for the obtention of construction materials etc., should be established with scales of 1 : 500 (for details) and to 1 : 10 000 (for general layouts).

4.6 Construction materials

The engineer geologist will determine the location, the quantities and the engineering characteristics of the construction materials; samples will be taken for analysis in a local laboratory, in order to determine its parameters and to verify its suitability.

4.7 Geological Conclusions

It is convenient to provide as soon as possible a report dealing with the investigations and justifications of the technical selection of the project scheme and the location of the main works with possible alternatives.

Apart from reporting on the topographic conditions, the report should include data relating to the structure and quantity of the rock and foundation materials, as well as describing the type of foundations must be recommended in view of the existing conditions.

Further recommendations should be given concerning: drainage works, desecation of the soil for structures foundation, and any special precautions to be taken during the process of construction including recomendations for execution and control.

CHAPTER 5

5. HYDROLOGY AND POWER ANALYSIS

5.1 Scope

This chapter describes the studies related to hydrology and power analysis. It consists of four parts which cover the topics of general hydrology, flow duration curves, energy production, and the computation of the main parameters concerning power generation.

The first part provides a general description of the mechanisms involved in hydrology and of the mathematical tools available to quantify the analysed phenomena. The second part deals in some details with the flow duration curve and the available methods to establish them in practical situations. Part three explains and defines the main parameters playing a role in the production of energy. Finally part four presents procedures to compute the variables which are to be considered for estimating the benefits of a power plant.

5.2 Hydrology, a quick overview

5.2.1 The hydrologic cycle

Hydrology, in its broadest sense, is the science which is concerned with the origin, distribution and properties of the waters of the earth. As such, it can be seen as the scientific examination and appraisal of the movement of water in the environment.

The concept of the hydrologic cycle is a useful tool to study hydrology. At the beginning of this cycle, water is evaporated from the oceans and the land. As water vapour, it is lifted into the atmosphere and transported by moving air masses until it finally falls as precipitation, either on land or in the oceans.

A part of the precipitation is caught by plants, and another runs over ground surface,both parts of the precipitation reach directly a stream or infiltrate into the soil. The infiltrated water either percolates to deeper zones to be stored as groundwater wich later flows out from springs or seeps into streams. During all the above mentioned stages of the water cycle, water will eventually evaporate into the atmosphere, and the cycle is completed.

Thus water undergoes various complicated processes of evaporation, precipitation, interception, transpiration, infiltration and seepage before it reaches a stream. Accordingly the four basic phases of interest to the hydrologist are: precipitation, evaporation and transportation, groundwater and surface streamflow. These topics are discussed hereafter.

5.2.2 The water balance

The drainage basin is a common unit of study used by the hydrologists. It may be defined as the area of land from which precipitation finds its way to a given cross-section of a river. On the other hand, runoff is that part of the precipitation as well as any other flow contribution, which appears in a surface stream. Hence runoff is the flow collected from a drainage basin and it appears at the outlet of this basin.

In a drainage basin, precipitation is converted to runoff by a series of processes which store and transmit water over the ground surface and within the various soil layers. The mode of operation of a drainage basin can be examined by analysing the relationships existing between the basin input and output.

The simplest input-output analysis is the water balance which assumes that the total volumes of water input and output are equal if no cumulative change of water held in the basin occurs. The balance equation states that:

$$\text{Precipitation} = \text{Evaporation} + \text{Runoff} \qquad (5.1)$$

These three variables are often measured in different units (precipitation and evaporation as average depth and runoff as volume). Hence, to be comparable, the total runoff volume must be divided by the area of the drainage basin and expressed in milimeters which then represents the equivalent average depth of water over the basin which forms the runoff.

In most cases, where the period of consideration has a finite duration, the volume of water stored in the basin may change and the balance equation must be modified in the following way:

$$\text{Precipitation} = \text{Evaporation} + \text{Runoff} \pm \text{Change in Storage} \qquad (5.2)$$

Due to difficulties in estimating the variable-basin-storage, it has become common practice to compute water balance on the basis of minimum storage. This has led to the definition of the water year, which usually starts when the ground and surface storage have both reached their minimum.

A useful parameter in hydrologic studies is the runoff coefficient which is given by the following relation:

$$\text{Runoff Coefficient} = \frac{\text{Runoff}}{\text{Precipitation}} \tag{5.3}$$

It represents that portion of the precipitation over a drainage basin which reaches the stream at its outlet.

5.2.3 The Runoff

From the hydrologic point of view, two major factors influence the runoff from a drainage basin, namely climatic factors and physiographic factors.

The climatic factors include mainly the effects of:

- ◇ Precipitation : form (rain, snow,...), intensity, duration, time and area distribution, frequency of occurrence, antecedent precipitation, etc.
- ◇ Interception : vegetation cover, seasons of the year.
- ◇ Evaporation : temperature, wind, atmospheric pressure
- ◇ Transpiration : temperature, solar radiation, wind, humidity, soil moisture, vegetation type.

The physiographic factors are :

- ◇ Basin characteristics : size, shape, slope, orientation, elevation, land cover and use, geology and soil type, topography (presence of lakes and swamps).
- ◇ Channel characteristics : slope, size, shape and roughness of cross-section.

Over a longer period of time, the runoff depth of a drainage basin is a function of precipitation and evaporation. These two components depend themselves heavily on the prevailing macro-climate and are practically independent of the basin, although some variations in precipitation with local topography may occur and evaporation rates are influenced by vegetation and soil cover.

Over time periods of less than one year, the basin runoff may show systematic variations resulting from the seasonality of precipitation and evaporation. As a consequence, there are as many typical flow regimes as there are major world climates. These regimes may show single or multiple maxima, be characterized by more or less extreme high and low flows, and experience shorter or longer periods with low flows.

As the time scale decreases, the effect of basin storage becomes increasinly important. The volume of water held in storage depends essentially upon the geology, soil type, topography (presence of lakes and or swamps) and precipitation itself.

The preceeding lines illustrate the complexity of the runoff phenomenon; adequate tools to analyse it are therefore required.

5.2.4 Characterization of the variability of runoff

The great importance of water to man's existence coupled with the natural variations of available water have induced man to measure and to record river discharges.

This is usually done with the help of a permanent structure where the water level in the river (named also stage or gauge height) is recorded either in a discontinuous, or preferably in a continuous way. With the help of a so called rating curve, which relates water levels to river discharges, the discharge corresponding to a given water level can be computed.

This operation is repeated with all the measurements made on a given day. The computed discharges are then averaged to obtain the average flow from the day. This figure is tabulated for each day of the year and, together with some other information, is included in a series published by the agency which has the responsibility to carry out this work. A typical presentation of such data is given in Table 5.1.

A graph showing discharge or flow as a function of time is known as a hydrograph. Time may be shown in minutes, hours, days or other units, depending on the purpose of the study, and discharge is generally shown in m^3/s. Figure 5.1 represents the hydrograph of a river for the year 1980.

The hydrograph can be considered as an integral expression of the climatic and physiographic characteristics that control the relations between precipitation and runoff of a particular drainage basin. It shows the variation in time of runoff at the point of measurement and represents the complexities of the runoff mechanism with a single curve.

Another graphical representation of the variability of runoff is the flow duration curve. It is obtained when the recorded discharges are arranged in the order of their descending magnitude and when for each magnitude the amount of time (days or percent) is computed, during which a given discharge is equalled or exceeded. A plotting of the magnitude of discharge as ordinate against the corresponding time scale (days or percent) as abscissa yields the duration curve (see Fig. 5.2). From a statistical point of view, a duration curve is a cumulative frequency curve of a time series, showing the relative duration of various magnitudes of discharge.

The length of the observation period used to establish the duration curve should preferably be one complete hydrologic year or a multiple of it. One should bear in mind that the shape of a flow duration curve may change as the number of hydrologic years considered increases.

The slope of the duration curve indicates the relative variability of runoff. If the curve is steep, the flows vary within a wide range. One can also quantify the shape of the duration curve in a variety of ways and use these shape parameters as analytical tools. The curve may either reflect the variability of the prevailing climate or, for basins with the same climatic characteristics, may indicate the effect of the basin variables or the storage capacity.

Statistics are very useful to study the variability of runoff. If enough data are available, mean values can be computed for annual, monthly and daily discharges. The determination of the related standard deviations and coefficients of variation supplies interesting indications on the variability of runoff. A further step is then the fitting of adequate statistical distribution curves to the available data sets. With the fitted distribution curves, it is possible to derive probabilities with which a given discharge is equalled or exceeded.

The same type of approach applies also for the analysis of low streamflows. In this case, the related studies should cover the severity, frequency and duration of the period of low flows. For more information on this as well as the preceding topics, the reader is referred to the specialized technical literature.

TABLE 5-1 SAMPLE PAGE OF A HYDROLOGIC YEARBOOK

Catchment area: 80 km2
Period of gaging: 1963-1980
Year: 1980

AVERAGE DAILY FLOWS

Day	JAN m3/s	FEB m3/s	MAR m3/s	APR m3/s	MAY m3/s	JUN m3/s	JUL m3/s	AUG m3/s	SEP m3/s	OCT m3/s	NOV m3/s	DEZ m3/s
1	0.20	0.16	0.16	0.16	0.28	0.71	1.39	2.63	1.35	0.76	0.43	0.22
2	0.20	0.16	0.16	0.16	0.29	0.67	1.36	2.86	1.17	0.76	0.40	0.22
3	0.19	0.16	0.16	0.16	0.29	0.66	1.30	3.08	1.15	0.77	0.39	0.22
4	0.19	0.16	0.15	0.16	0.30	0.85	1.24	3.02	1.18	0.72	0.37	0.21
5	0.19	0.17	0.16	0.17	0.31	1.01	1.17	3.30	1.27	0.74	0.36	0.21
6	0.19	0.16	0.16	0.17	0.32	1.19	1.17	3.47	1.24	0.67	0.36	0.21
7	0.18	0.16	0.16	0.17	0.34	1.36	1.24	3.25	1.13	0.62	0.34	0.20
8	0.18	0.17	0.16	0.16	0.36	1.43	2.00	3.64	1.19	0.57	0.33	0.20
9	0.18	0.18	0.14	0.16	0.37	1.53	2.45	3.20	1.14	0.48	0.32	0.20
10	0.18	0.17	0.14	0.17	0.35	1.63	2.05	2.77	0.99	0.46	0.31	0.20
11	0.18	0.16	0.14	0.17	0.38	1.71	1.77	2.19	1.05	0.44	0.30	0.19
12	0.18	0.15	0.14	0.17	0.49	1.90	1.53	2.01	1.06	0.43	0.29	0.19
13	0.17	0.15	0.14	0.17	0.55	2.29	1.40	2.00	1.10	0.41	0.27	0.19
14	0.17	0.15	0.14	0.17	0.63	2.65	1.44	1.91	0.97	0.40	0.27	0.18
15	0.17	0.15	0.13	0.17	0.64	2.99	1.75	2.02	0.97	0.40	0.31	0.18
16	0.17	0.15	0.13	0.17	0.58	3.16	2.21	2.30	1.05	0.46	0.28	0.18
17	0.17	0.15	0.13	0.17	0.51	2.80	2.07	2.31	1.03	1.34	0.27	0.18
18	0.17	0.15	0.13	0.18	0.46	2.27	1.99	2.02	1.05	0.75	0.26	0.18
19	0.17	0.15	0.15	0.18	0.43	2.25	2.10	2.00	1.11	0.65	0.25	0.17
20	0.16	0.15	0.15	0.19	0.42	2.22	2.22	2.05	1.04	0.58	0.26	0.17
21	0.17	0.15	0.15	0.19	0.46	2.23	2.52	2.17	1.00	0.54	0.24	0.17
22	0.17	0.15	0.15	0.20	0.53	2.27	1.91	2.16	1.10	0.53	0.24	0.17
23	0.16	0.14	0.15	0.21	0.60	2.16	1.92	1.90	1.30	0.49	0.23	0.17
24	0.16	0.15	0.15	0.22	0.61	2.09	2.09	1.72	1.05	0.47	0.23	0.17
25	0.15	0.16	0.15	0.22	0.62	1.81	2.22	1.52	0.99	0.42	0.23	0.17
26	0.16	0.16	0.15	0.24	0.71	1.57	2.48	1.44	0.96	0.42	0.23	0.17
27	0.15	0.17	0.16	0.24	0.74	1.43	2.74	1.33	0.94	0.43	0.23	0.17
28	0.15	0.16	0.18	0.25	0.77	1.32	2.88	1.38	0.91	0.45	0.23	0.16
29	0.16	0.17	0.17	0.26	0.80	1.35	2.74	1.46	0.90	0.46	0.23	0.16
30	0.15		0.17	0.27	0.78	1.37	2.77	1.62	0.81	0.46	0.23	0.16
31	0.15		0.17		0.74		2.64	1.56		0.44		0.16
AVERAGE FLOW:	0.17	0.16	0.15	0.19	0.51	1.76	1.96	2.27	1.07	0.57	0.29	0.18
MAX. FLOW:	0.20	0.18	0.18	0.27	0.80	3.16	2.88	3.64	1.35	1.34	0.43	0.22
MIN. FLOW:	0.15	0.14	0.13	0.16	0.28	0.66	1.17	1.33	0.81	0.40	0.23	0.16
ANNUAL AVERAGE:		0.70										
PERIOD 1963-1980:												
AVERAGE FLOW:	0.13	0.12	0.13	0.25	0.81	1.66	1.89	1.61	1.05	0.57	0.30	0.18
MAX. FLOW:	0.30	9.90	1.00	0.90	2.70	4.40	10.50	8.90	3.70	4.70	1.50	0.40
MIN. FLOW:	0.09	0.08	0.09	0.09	0.11	0.64	0.69	0.71	0.40	0.24	0.14	0.11
ANNUAL AVERAGE:		0.73										

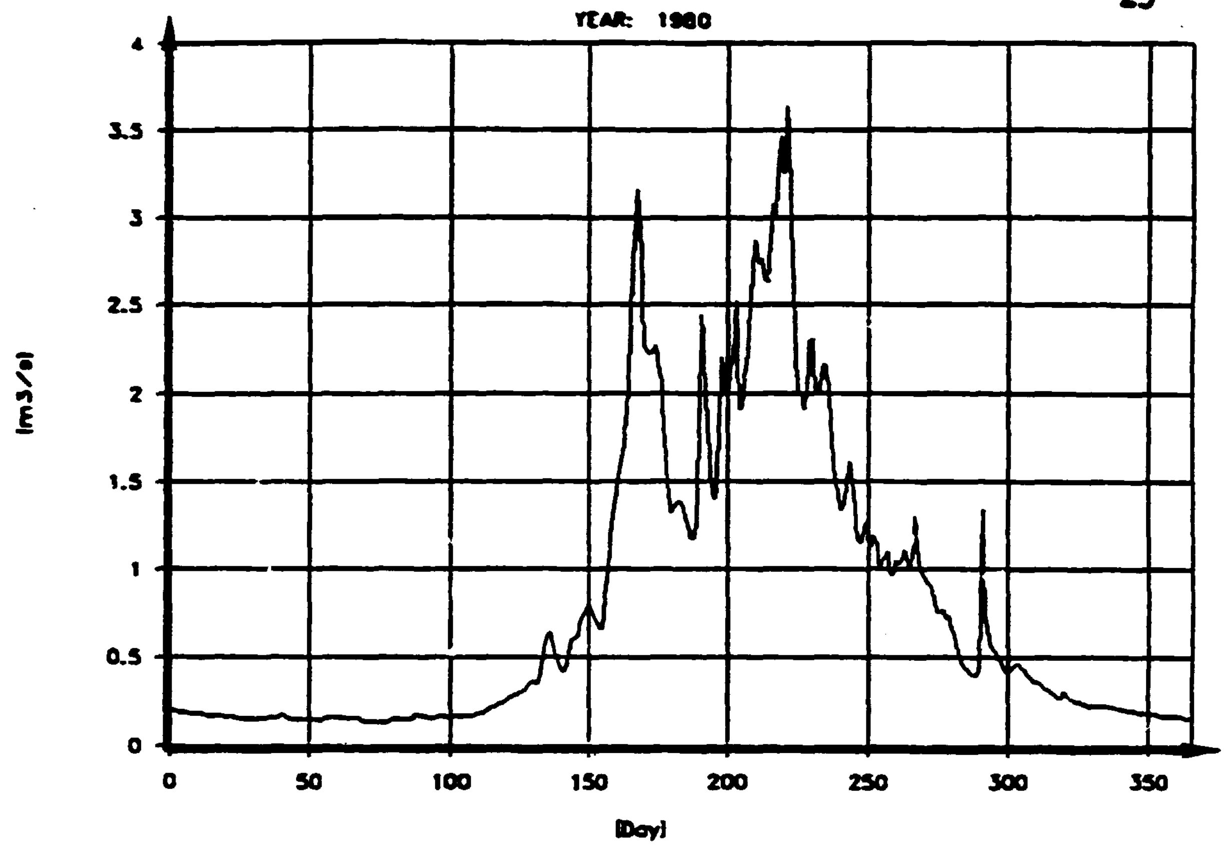


Fig. 5.1 Hydrograph

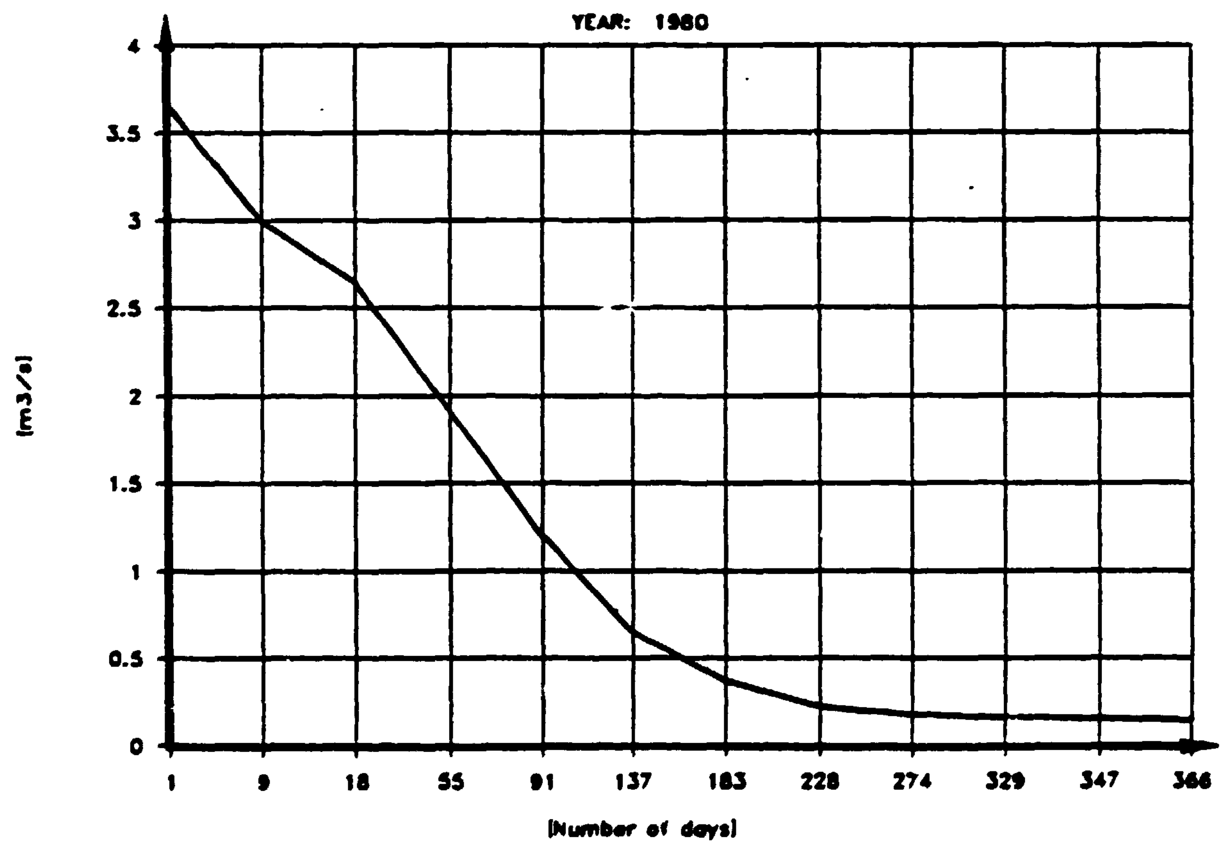


Fig. 5.2 Flow Duration Curve

5.2.5 Floods

In a period of heavy precipitation, rivers naturally experience high discharges. Often the river channel cannot accomodate the related peak discharge. A flood is defined as a flow in excess of channel capacity; it is a normal and expected characteristic of any river.

The simplest and most complete way to characterize a flood is to draw the corresponding flood hydrograph (see Fig. 5.3). The main components of a flood hydrograph are the rising limb, the peak discharge and the recession. In the rising limb, discharge increases rapidly until the peak discharge is attained. This point is reached when the quantity of water draining through the gauging station from the basin has reached a maximum. It usually takes place shortly after the rain has ceased. Thereafter follows the recession, during which the amount of storage water in the soil and in the bedrock controls the discharge. Hence the shape of the recession limb is given by the rate of withdrawal from storage. Peak or instantaneously discharge, flood volume and flood duration are the main characteristics of a flood.

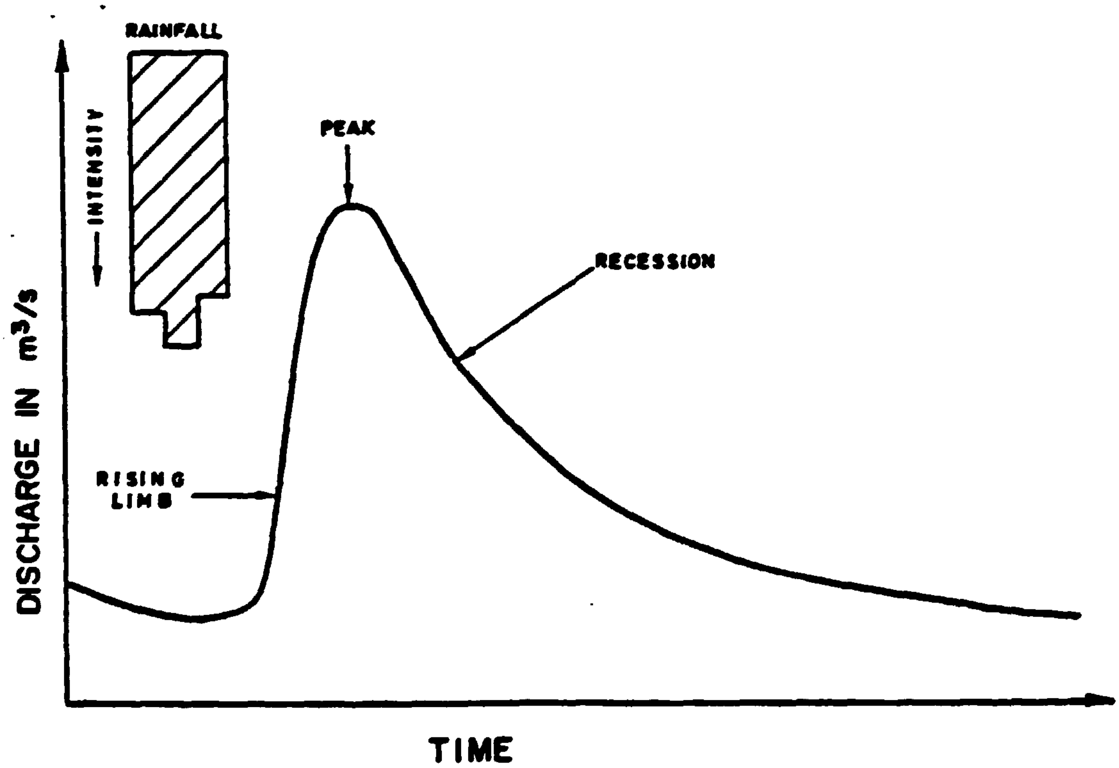


Fig. 5.3 Components of a Flood Hydrograph

A variety of factors, many being interrelated, control the shape and dimensions of the flood hydrograph. These factors can be broadly divided into two main categories. The first category include climate related features like storm characteristics (precipitation, intensity, duration, and total amount), interception and detention, evaporation, infiltration and storage capacity. The second one includes those elements which are of permanent nature like drainage basin (area, shape, elevation, slope, drainage network, channel characteristics (cross-section, roughness, slope), vegetation and land use.

The instantaneous peak discharge is one of the most important parameters needed for design of structures along or across a river. There are several ways to compute it. The method which is finally retained depends on the availability of data, the type of structure concerned and the characteristics of the river regime.

If enough data are available, flood frequency analysis should be performed. This method begins with the tabulation of the highest instantaneous or mean daily discharge in each year of record at the gauging station. The set of data obtained are arranged in order of magnitude and a recurrence interval or frequency factor is attached to each selected event. These pairs of points can then be plotted on special graphical paper and a statistical distribution curve, adjusted to it. Finally it is possible to compute with the fitted statistical distribution curve the peak instantaneous (daily) discharge corresponding to any recurrence interval.

In case not enough runoff data are available, other methods to compute peak discharges exist, based either on precipitation records or on maximum values gained from experience. In the first case, (availability of precipitation records), formulas like the rational formula and the maximum probable flood can be utilized. In the second case (maximum values), various envelop curves, like the ones of Meyer-Jarvis and of Creager can be applied. However, the application of these formulas should be left to the specialists.

5.3 The Flow Duration Curve

5.3.1 Generalities

Now that the reader is familiar with the basic theoretical principles of hydrology, it is time to deal with the practical problems.

Hydrologic data are needed:

- to select the installed capacity.
- to compute the energy production and the related dependable capacity
- to perform the economic analysis

◇ to prepare a safe design.

The determination of the parameters related to the energy production is best done on the basis of the flow duration curve. Accordingly, the emphasis of the hydrologic studies lies in the computation of the flow duration curve.

5.3.2 Daily runoff records are available

If a gauging station has been in operation for at least several years in the vicinity or at the site of the planned diversion point, the preparation of the average duration curve is straightforward. However, before performing any computation, one should check the quality of the available data.

First to be checked is the computation of the daily discharges from the measured gauge heights (use of the correct relation between gauge height and discharge, stability of this relation over the years) and the evaluation of the extreme flows (both minimum and maximum). Secondly, it should be ascertained whether the gauging station has measured in the past and still measures the flows which are really available to the power plant. As a third step it has to be made sure that the available discharge data are homogeneous during the period of records (existence of trends). Finally, a complete list of missing data must be established and their importance should be evaluated.

If during the mentioned controls, errors, trends and/or gaps have been set forth, the available discharge data must be corrected, adjusted and/or completed. Only when all these operations have been carried out satisfactorily, can computation be started and flow duration curve as indicated on Fig. 5.2 can be established.

5.3.3 Daily runoff records are not available

Unfortunately either incomplete or no records at all are available for many projects. The computation of the required duration curve may then become very tricky, and it is suggested that in these situations a qualified hydrologist be consulted.

The procedure to be followed differs from case to case, depending on the type of available discharge data, nature of the river regime , and characteristics of the project. However, in order to enable some preliminary assessments to be made and better understand the methodology adopted by the hydrologist, the hereafter described approach is indicated.

This general approach proceeds stepwise. First, the average annual discharge or runoff is computed. This variable is generally the most important one and fortunately also the easiest one to determine. It is recommended to define for this variable also the upper and lower limits. Thereafter, the distribution of the average annual runoff over the twelve months of the year is estimated. Thirdly, the duration and severity of the period of low flows is evaluated. Finally the required flow duration curve is constructed on the basis of the obtained results.

The type of data and procedures used in the compilation of the flow duration curve depends to a great deal on the problem at hand and on the type and amount of average hydrologic data available. These procedures make heavy use of the relations existing on the one hand between runoff and precipitation in a given drainage basin, and on the other hand between runoffs of nearby or similar drainage basins. Broadly speaking, they fall into the following three categories:

In the first one, the general data situation is rather poor. For example only precipitation data are available for the basin under study, and a few concurrent precipitation and runoff records exist in a basin nearby or similar to the one under consideration. In this case, a great freedom is left to the hydrologist and the analysis to be carried out is mainly qualitative. He will work, with the concept of the runoff coefficient and, drawing heavily on his experience, select that set of values which least contradict the available information.

In the second one, the data situation is better. Quite generally, more runoff data are available and the hydrologic characteristics of the various basins under consideration lie closer to one another. This improved situation allows the application of quantitative methods like correlation analysis. The pairs of variables correlated maybe, either runoff and precipitation in the project basin, and/or runoff measured both in the project and in the nearby basin.

In the third one, the data situation and the catchments being considered are such that conceptual precipitation-runoff models can be used. The selection of an adequate precipitation-runoff model and its calibration represent the most important and delicate tasks. Once these operations have been satisfactorily completed, discharge data can be obtained and the required flow duration curve computed.

5.4 Energy Production : Definitions and Formula

5.4.1 General

Installed capacity, energy production and dependable capacity are the main parameters to be selected. Before the approach to be followed to compute these parameters is described in details, some basic formula and definitions are presented hereafter.

Energy is created as a result of water falling between two levels. Power is the rate of energy generation. The following equation may be written :

$$P = 9.81\ Q \cdot h \cdot e \qquad (5.4)$$

where P = power generated, kW
Q = flow of water through turbines, m^3/s
h = net head on turbine, m
e = power plant efficiency

During the process of energy generation, a portion of the potential energy is lost. The losses associated with the generators and transformers are rather small. The ones associated with the turbines are much greater and depend on the turbine type and on the operating conditions. Typically the efficiency of a turbine varies from 70 to 90 percent. To obtain the energy output, it is usually sufficient, to assume an average value for the power plant efficiency. As a reasonable first approximation, a value of 0.85 is recommended.

5.4.2 Head

The term head means the vertical distance through which the flow to the power plant falls. One must distinguish between the following kinds of head.

The headwater elevation means the elevation of the water surface in the forebay structure (intake), from which the releases are made to the powerplant. Depending on the type of powerplant, the water surface may remain practically constant(canal drop) or vary (reservoir).

The tailwater elevation means the elevation of the free water surface immediately downstream of the powerhouse. Depending on the hydraulic characteristics of the tailrace, respectively of the river, this elevation may remain practically constant or vary.

The static or gross head is defined as the difference between the headwater and the tailwater elevation. It represents the theoretical head which would be available to the turbines, if there were no losses in the waterways.

Under head losses one understands the losses of head which occur between the intake and the tailrace of the powerplant. They result from friction and other disturbances within the waterways.

The effective or net head is given by the differences between the static head and the head losses. It represents the head available for power generation. Depending on the type of scheme considered, the effective head either stays practically constant or varies. In the second case, the average effective or net head should be computed and introduced into the energy computations.

The design head is the effective or net head at which peak efficiency is obtained. It should be selected in such a way that the maximum and minimum heads do not fall outside the permissible operating range of the turbine. As a first approximation, the operating head should be comprised between 60 and 120 percent of the design head.

Finally the rated head is the effective or net head at which the full gate output of the turbine produces the generator rated output.

5.4.3 Flows

The flow or discharge passing through the turbine is another key element for the determination of the energy production of a power plant. As for the head, various types of flow can be defined.

In some cases, when the discharge in the river exceeds a given high level, the plant has to be shut down, in order to avoid critical operation conditions for the powerplant.

The maximum turbine flow is the largest flow which can be absorbed by the turbine. This maximum flow is related to the design flow. For almost all turbines, the maximum flow can be taken equal to about 115 percent of the design flow.

A turbine is designed to have a capacity for a specific net head (design head) at which it reaches its peak efficiency. The maximum flow at this head is called the design discharge, (see Fig. 5.4).

There is a minimum flow below which a turbine cannot be operated safely or efficiently. This minimum flow amount to about 30 to 50 % of the design discharge, (see Fig. 5.4).

The rated capacity is obtained with the following formula :

$$P = 9.81 \cdot Q \cdot h \cdot e \qquad (5.5)$$

where P = rated capacity, in, kW
Q = discharge at the rated head, in m3/s
h = rated head, in m
e = efficiency of the powerplant.

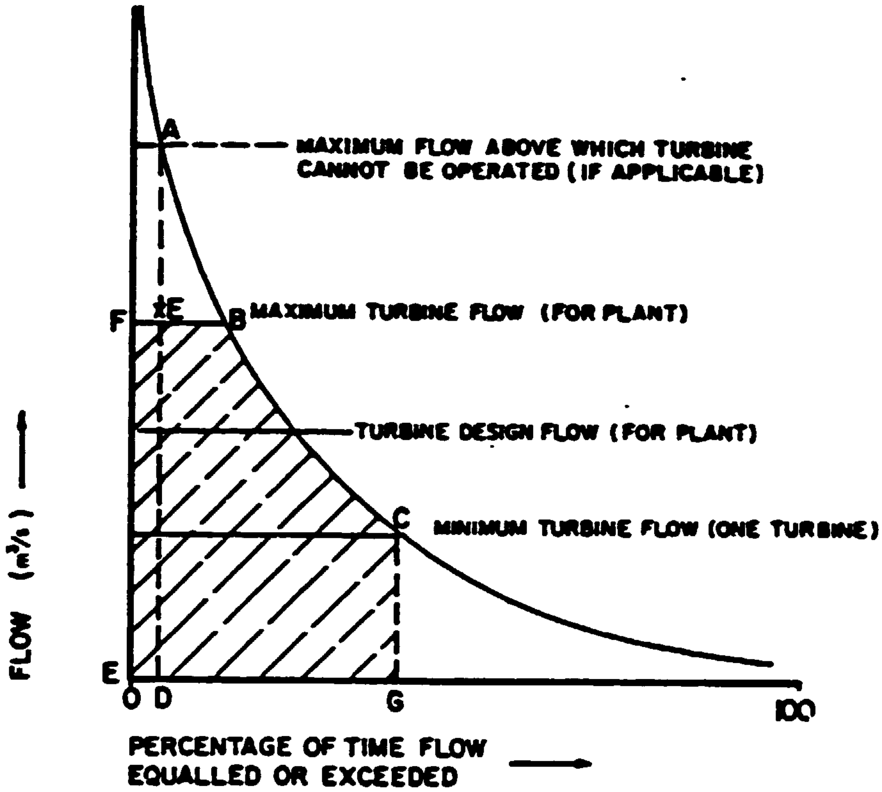


Fig. 5.4 Flow Duration Curve and Energy Production

The plant factor is the ratio of the average annual energy actually generated by the plant to the energy which could be generated if the plant were operated at full capacity for the entire year.

5.5 Capacity and Energy Computations

5.5.1 Plant Capacity

Generally the plant capacity is not known at the beginning of the study and has hence to be determined. The selection of the plant capacity is performed in an optimization process.

The procedure is an iterative one. A range within which the optimal plant capacity is expected to lie is defined, together with a few plant capacities within this range. It is then assumed that the plant capacity is increased from the lowest to the second lowest value, and the related incremental costs and incremental power benefits (energy and capacity) are computed. If the incremental benefits exceed the incremental costs, the plant capacity is increased, and the same computations are carried out with the following pair of capacities. This procedure is repeated until the point is reached where the incremental costs just equal the incremental benefits. This point represents then the preferred capacity.

The optimization process is theoretically straightforward. There are however some pitfalls, especially concerning the valuation of the incremental benefits, which should be avoided. It should be verified that the incremental energy production and that the incremental capacity can effectively be absorbed by the system (not all the power plants can operate together in the peaking zone). Furthermore, it is not unusual that the value of energy is lower during the period of high flows.

With the construction of numerous power plants, considerable experience has been gained in the field of the selection of the plant capacity. This great experience has allowed to derive rules of thumb. According to them, the optimal plant capacity generally lies within a range defined on the flow duration curve by the 35% and 25% excedance values. Also it is generally economically not feasible to develop a plant for a maximum flow which is exceeded only 15% of the time.

5.5.2 <u>Energy Production</u>

The estimation of the average annual energy production of a power plant is best explained with the help of the flow duration curve shown on Fig 5. The procedure is as follows :

◇ Define the points **A**, **B**, and **C** on the flow duration curve, which corresponds to the maximum flow above which the turbine cannot be operated (if applicable), the maximum turbine flow and the minimum turbine flow.

◇ Draw vertical straight lines through the points **A** and **C**, and determine the points **D** and **G** which are the intersection of these lines with the horizontal axis.

◇ Draw a horizontal straight line through the point **B**. The intersection of this line with the line **AD** and with the vertical axis gives the points **E** and **F**.

Assume first that the powerplant can be operated regardless of the flow in the river. Then the area bounded by the two axes, by the straight lines **BF** and **CG**, and by the portion **BC** of the flow duration curve represents the average annual volume of water available to generate energy.

If the powerplant has to be shut down during periods of high flows, the resulting loss of water for the powerplant is given by the area bounded by the points FEDO.

The average annual discharge Q (m^3/s) available can be deduced easily from the average annual volume just computed. The average annual energy production is then computed as follows :

$$E = 9.81 \cdot Q \cdot h \cdot e \cdot 8760 \qquad (5.6)$$

with E = average annual energy production (kWh)
Q = average annual discharge available to the powerplant (m^3/s)
h = average net head (m)
e = efficiency of the powerplant

Once the average annual energy production is known, the plant factor can be computed, using the definition given unter 5.4.3

5.5.3 Dependable Capacity

The value of the delivered power depends on whether or not a dependable capacity can be assigned to the plant. Quite often only a part of the capacity is dependable. The precise determination of this parameter is a complex procedure and cannot be dealt with here in details. However, in the following, a few general concepts will be presented, mainly to alert the readers to the problem.

Basically three elements play a role, namely the hydrology of the basin of the powerplant, the characteristics of the load of the network and the other generating plants in the system. If the plant operates in an isolated system, its dependable capacity is given by the minimum flow corresponding to a prescribed frequency. As part of a larger system, the dependable capacity is influenced by the characteristics of the other plants in the system and by the relations existing between the occurrence of the low flow period and of the peak loads of the network.

Obviously, the first task will be to define the characteristics of the discharges during the period of low flows (duration of that period and related discharges, variation of these parameters from year to year, statistical analysis). The results of these computations will show how critical this issue is and whether it is worth to embark on refined and complicated system analyses.

CHAPTER 6

6. THE WATER INTAKE

6.1 Generalities

In this chapter, we will consider water intake features of run-of-river schemes and drop intakes (also known as Tyroler type intakes); schemes with dams and reservoirs are beyond the limited scope of this report, and are unusual for small hydroplants.

Water intake requires the construction of a diversion structure, sometimes with a weir or a small dam across the river. It goes without saying that a river can be tapped anywhere along its length (see Figs. 6.1 and 6.2). This stage of the design however requires a very carefull study in order to avoid future problems and eventually the total failure of the main function of the water intake or its total collapse.

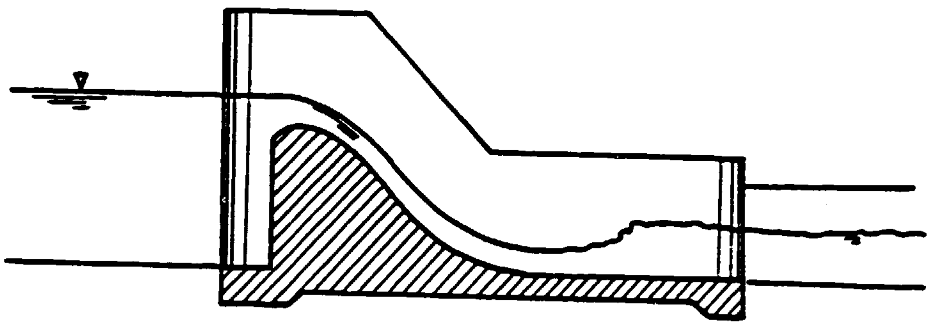


FIG. 6.1 Schematic free flow type intake

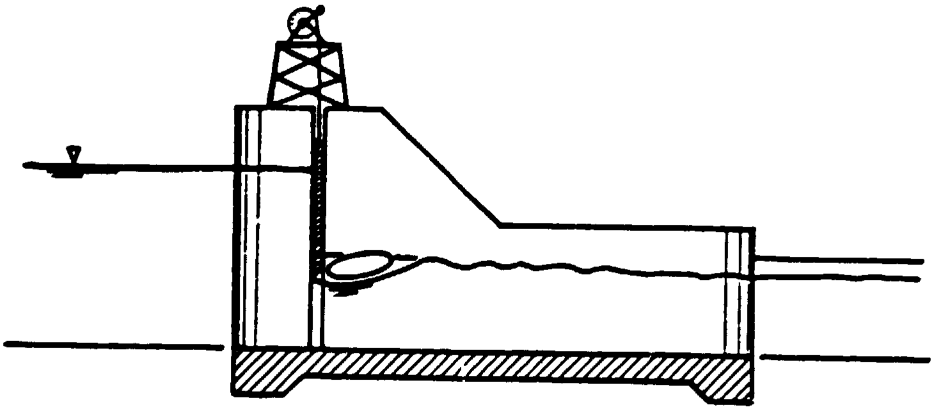


FIG. 6.2 Schematic submerged flow type intake

For the location of the intake structure, the engineer in charge of the design should consider the following factors:

- Topographic conditions
- Geological and geotechnical conditions along with the nature of the stream bed
- The natural bends along the river
- Water rights and water used for other purposes
- The accesibility to the selected intake location.

6.1.1. Topographic Conditions

The topographic conditions should be carefully studied, since they will determine the type of weir and the type of intake that can be developed. The possibility of slope instabilities in the inmediate vicinity of the structures should be investigated, in order to avoid such areas , or to protect them if those areas can not be avoided.

6.1.2 Geologic and geotechnical conditions

The geologic and the geotechnical conditions will determine the kind of foundation to be used for the weir and for the intake structures, if those structures can be made of a simple masonry, or if they require a more solid construction with reinforced concrete. The geological and the geotechnical conditions will determine too, the type of criteria to be used during the design stage of the structures.

6.1.3 Nature of the streambed

It is important that the elevation of the stream bed be maintained at the location of the intake in order to guarantee the proper functioning of the intake. In most cases the stream bed transports sediments and granular material, and presents the latent danger that erosion might occur. The streambed may be significantly eroded and lowered below the elevation of the intake entrance, preventing the desired water from entering the intake (see Fig. No. 6.3).

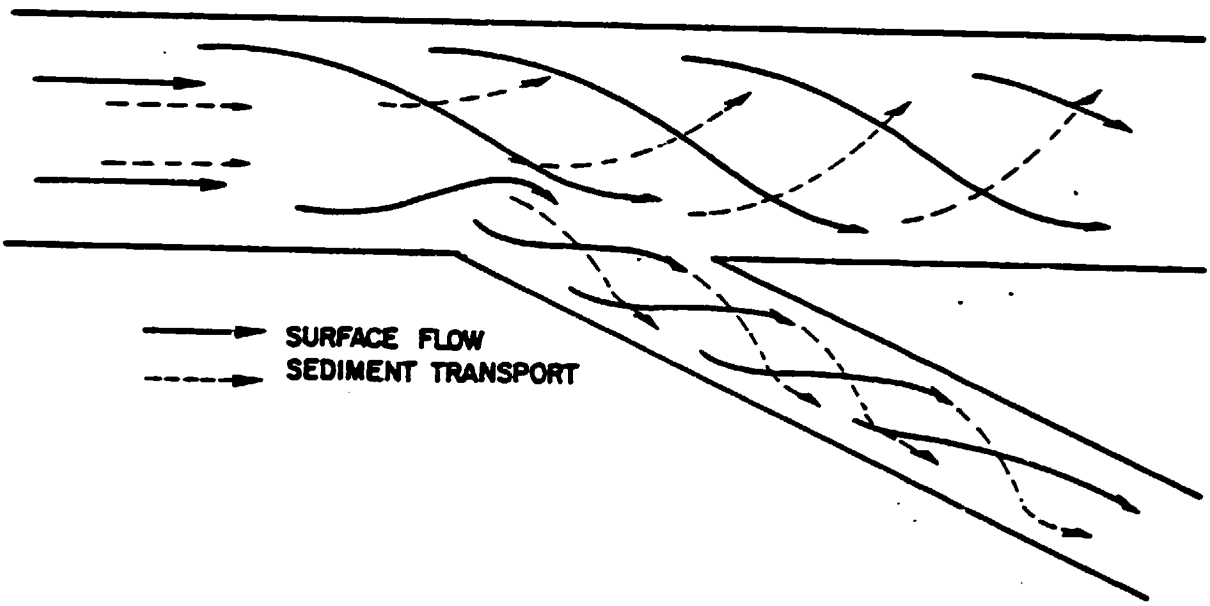


FIG. 6.3 Schematic stream flow on a straight section of a river with a lateral branch.

This situation should and can be avoided with the construction of a weir, the location of the intake in a proper place, where the geological and geotechnical studies indicates that the stream bed will be stable and permanent (this effect can be obtain with a stream flow over bed rock, in a river with small slope or in a river with all-year-round constant flow).

However, in most cases erosion as well as sediment transport are present, and a permanent weir will be required to be able to maintain the level of the river bed relatively constant at the intake.

A disadvantage of this system is that a weir constructed on a river bed with sediment transport can obstruct or damage permanently an intake, and that provisions have to be made to let the excedent of sediments pass across the weir or barrier without entering into the intake.

The selection of the site or location of the intake should be made in such a way that the above technical conditions can be met, at the same time taking into consideration that the costs should be minimized, a fact that depends on the structure size and volume, as well as the method of construction and simplicity of the structure to be built (see Fig. No. 6.4).

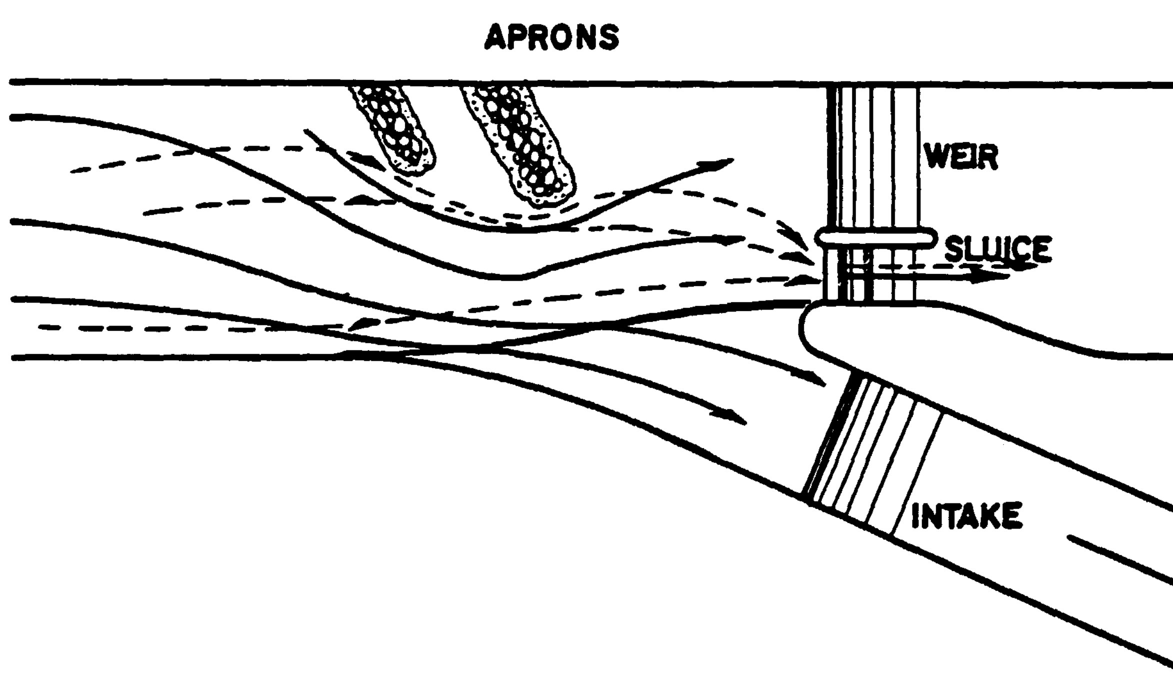


FIG. 6.4 Schematic correction of a straight river with the help of aprons

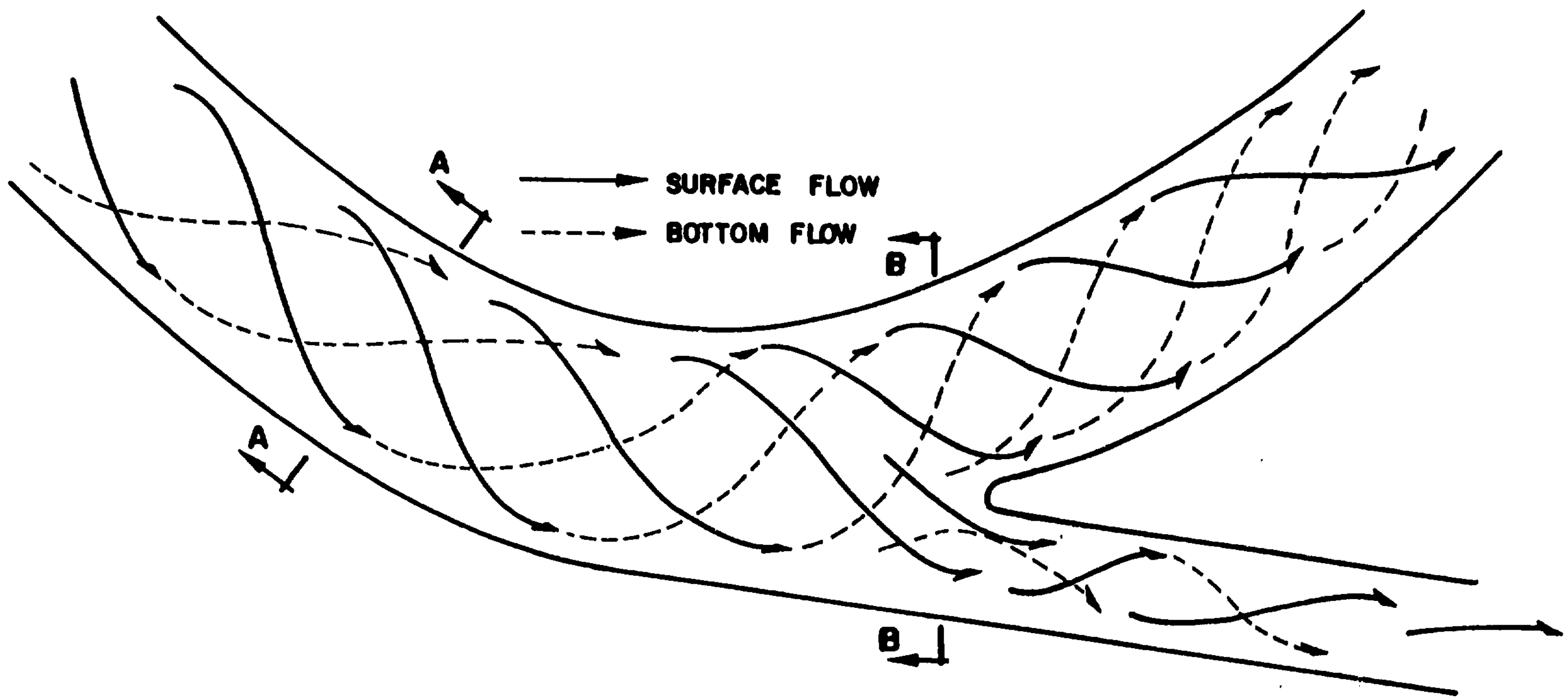


Fig. 6.5 Schematic stream flow on a curved section of a river

6.1.4 Natural bends of the river

The orientation and situation of an intake in a river can decide on its success or its failure in fulfilling its desired purpose. Advantage should be taken of the natural bends of the river, always placing the water-intake wherever possible at the outside bend or on a relatively straight section of a stream, with the entrance of the water intake oriented laterally and not oriented upstream of the water flow.

The transport of riverbed sediments is a main problem to be considered during the design phase of an intake structure. Because the surface stream flow moves in a bend of a river, laterally and faster than the stream flow on the bottom of the river, the water rotates on a bend like a corkscrew (see Figs. 6.5, 6.6, and 6.7), the sediments will move in a somewhat opposite direction. Floating materials such as wood, can be easily deviated, controled or removed with the help of trash racks. However, granular material and sediments transported by the stream flow have a tendency to form deposits in front of the intake entrance.

For that reason, it is convenient to create a device to retain the sediments first, and afterwards to channel them through a passage specially designed and constructed for such a purpose, allowing them to pass the barrier or weir and to maintain the front of the water intake clear of sediments. This operation should be conducted with a minimum of water losses; it can be done manually or mechanically, depending on the size and the importance of the structures,(see Fig. 6.8).

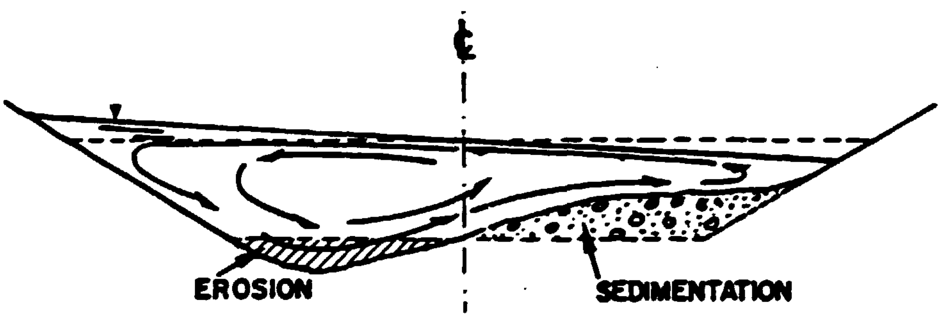


FIG 6.6 Schematic cross flow on a section A - A

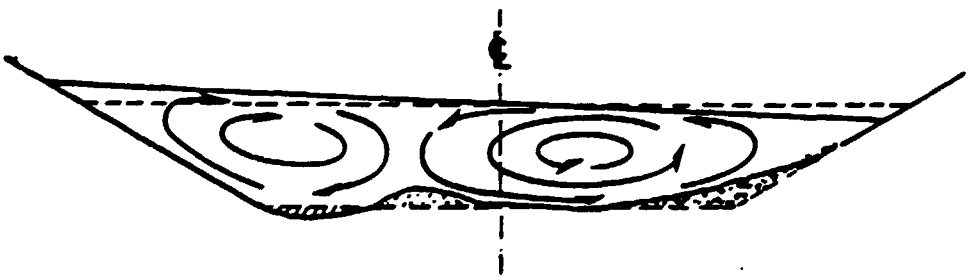


FIG. 6.7 Schematic cross flow on a section B - B

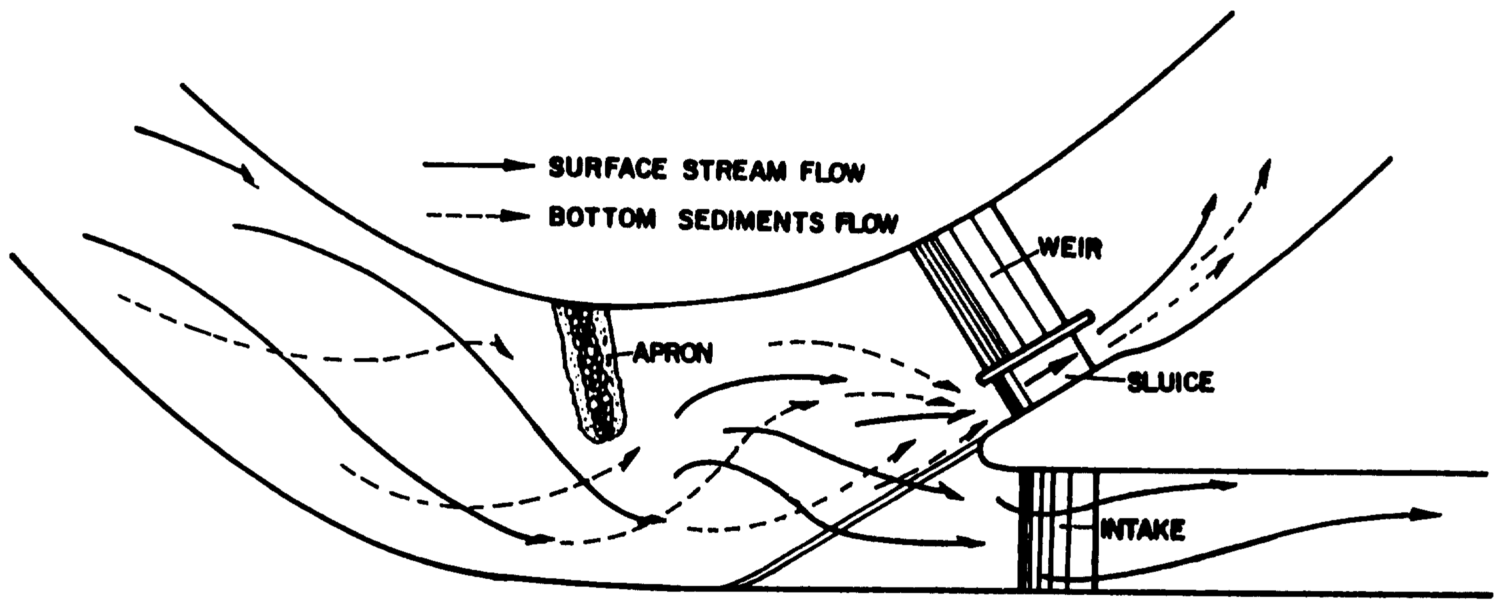


FIG. 6.8 Schematic Melioration of flow conditions at the intake in a river bend with an apron

Experience teaches us certain general rules which should be taken into consideration during the design of water intakes:

1. If possible, it is convenient to install the water intake in the outside elbow of a stream.
2. The water intake slowsdown the transport of sediments and the sharper the curve of the stream flow, the greater the slowdown process of the sediment transport will be.
3. Usually the most efficient location of the water intake is the lower end part of the curve, or in a straight section immediately after the end of the curve.
4. The experience demonstrates, too, that the design and lay-out of a water intake that take advantages of the above-mentioned conditions are the most efficient and successfull during the process of flushing and clearing the sediments deposited in front of the intake entrance.

6.1.5 Water rights and other water uses

It is important to clarify the legal implications concerning the present and the future water rights, before the owner launches any physical construction.

If other uses of water are envisaged or have been decided in the past, (such as water supply or irrigation), it may be possible that compromises could or should be made to satisfy the users with other purposes, trying to combine if possible those necessities. For example it should be possible to turbinate the water before it is to be used for irrigation or water supply purposes. Should this not be possible, allowances will have to be made and the intake will have to be complemented by means of with bypass facilities.

Rules and regulations will have to be established for the use of the water and its priorities, setting in advance the periods for energy generation. A carefully planned and coordinated schedule duly accepted by all potential users must be established.

If irrigation and power generation each require a substantial percentage of the available water, it might be possible to operate both users in parallel, generating power when water is not being used for irrigation purposes.

6.1.6 Accessibility to the selected intake location

All water intakes should be planned in such a way as to have a permanent access. The access to the project area should be developed and completed during the early stages of the project. A good access road during the construction period will not only facilitate the construction but will reduce costs as well.

After the construction of the project has been completed, it is recommended to keep the access road open and well maintained all year round.

6.2 RUN OF RIVER OR CONVENTIONAL WATER INTAKES

6.2.1 Intake Structure Design Considerations

The water at the intake, is divided into usefull water to be captured by the intake, and rest water to be passed over the weir or barrier.

Because the river transports not only water but sediments and debris as well, it is necessary to let part of the water pass the barrier and remain in the original river course. For power generation it is desirable to utilize water as clear as possible; the bottom sediments have therefore to be periodically flushed in order to keep the intake entrance free and unobstructed, an operation that requires a certain amount of water losses.

The hydrological study will determine the Q_{design} and will determine the amount of water to be allowed to pass through the barrier as unused water. The latter is determined by the quantity of water required to flush the sediments plus the water quantities that are required for other purposes as for example for water supply, for irrigation or for any other purpose, (see Fig. 6.9).

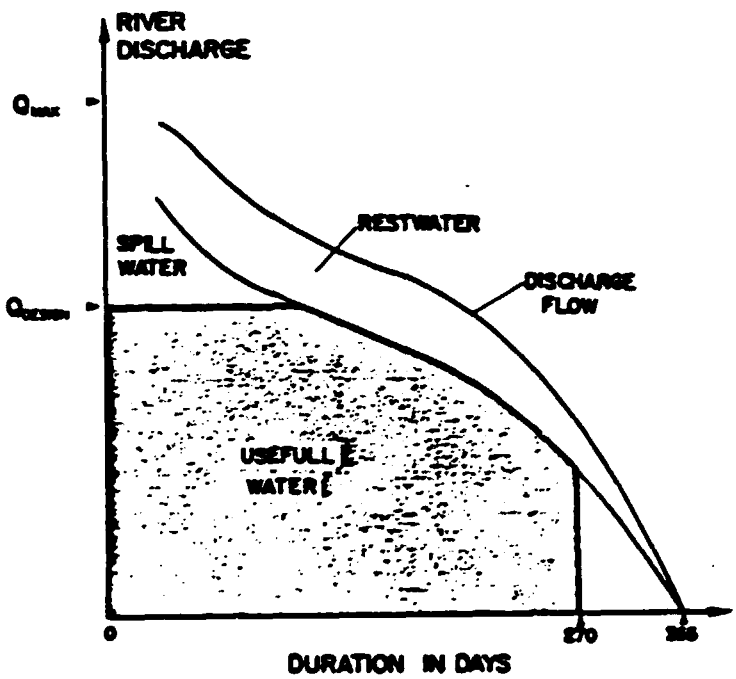


FIG. 6.9 Determination of Q_{design} with a flow-duration curve.

The Q_{design} or design discharge is the water discharge used to dimension the intake structures. It is the maximum water discharge allowed to be diverted from the main stream and it limits the amount of water diverted from the total incoming water during the wet periods.

The hydrological study has to take into consideration the water requirements for other uses and to determine their importance. An economic study will optimize such water uses and the value of Q_{design}.

6.2.2 The stream flow conditions in front of the water intake structure

In straight portions of the river, the stream flow on the surface and the sediment transport on the bottom of the river move parallel to each other However in the curved portions of the river or elbows the water on the surface has the tendency to flow more on the outside of the river elbow. the sediments on the bottom, on the contrary, will accumulate on the inside of the river elbow, with the natural consequence that the outside will be eroded and that deposits of sand gravel and other sediments will occur on the inside (see Figs. Nr. 6.5, 6.6, 6.7, and 6.8).

This fact if well studied can be used to the benefit of the scheme, in the sense to concentrate a maximum of clear water into the intake entrance. and to deviate the sediment transport of solids away fron the intake entrance and guiding them directly to the flushing gates. However this principle as simple as it appears, if not properly applied, can be influenced in the practice by minor factors that can alter the entire purpose of the intake ; this may eventually lead to a failure .

6.2.3 The main elements of a river water intake

The main elements of a river water intake are the following:

- The weir or barrier (can be fixed, mobil or of a mixed type).
- Intake entrance.
- Forebay.
- Flushing channel.

6.2.3.1 The Weir

The weir or barrier is usually build perpendicular to the flow direction of the river, its main purpose being the retention of the river water In many cases it can be used at the same time as a spillway structure to allow to spill the water excedents. In some cases because of the topographic or geologic restrictions it may be necessary to arrange the weir in a position that is not perpendicular to the stream flow of the river. This situation can be accepted provided that the economic restrictions as well as the technical requirements are dully fulfilled.

The weir has to comply with at least the following conditions:

- ◇. It should be designed to allow to pass the Qmax (a thousand years discharge).
- ◇. It should be calculated against overturning moments and against shear sliding with factors of safety not lower than 1.3.
- ◇. Erosion downstream or at the toe of the spillway should not be allowed. If this danger is latent, an energy dissipator or rip-rap protection should be provided.

The discharge of the water above a spillway or a weir is given by the following formula:

$$Q = 2/3 \cdot \mu \cdot L \cdot \sqrt{2g} \cdot [H^{3/2} - (v^2/2g)^{3/2}] \qquad (6.1)$$

when the approach velocity is small, this formula can be simplified into:

$$Q = 2/3 \cdot \mu \cdot L \cdot \sqrt{2g} \cdot H^{3/2} \qquad (6.2)$$

or

$$Q = C \cdot L \cdot H^{3/2} \qquad (6.3)$$

were: μ = Discharge coefficient (Dimensionless)

L = Width of the spillway or weir (m).

g = Acceleration of gravity (= 9.8 m/s^2).

H = Energy head above the spillway (m).

$$C = 2/3 \ \mu \cdot \sqrt{2g} \ (m^{1/2}/s). \qquad (6.4)$$

The coefficient μ, respectively the factor C fluctuate in between the following values:

TABLE Nr. 6.1
SPILLWAY COEFFICIENTS

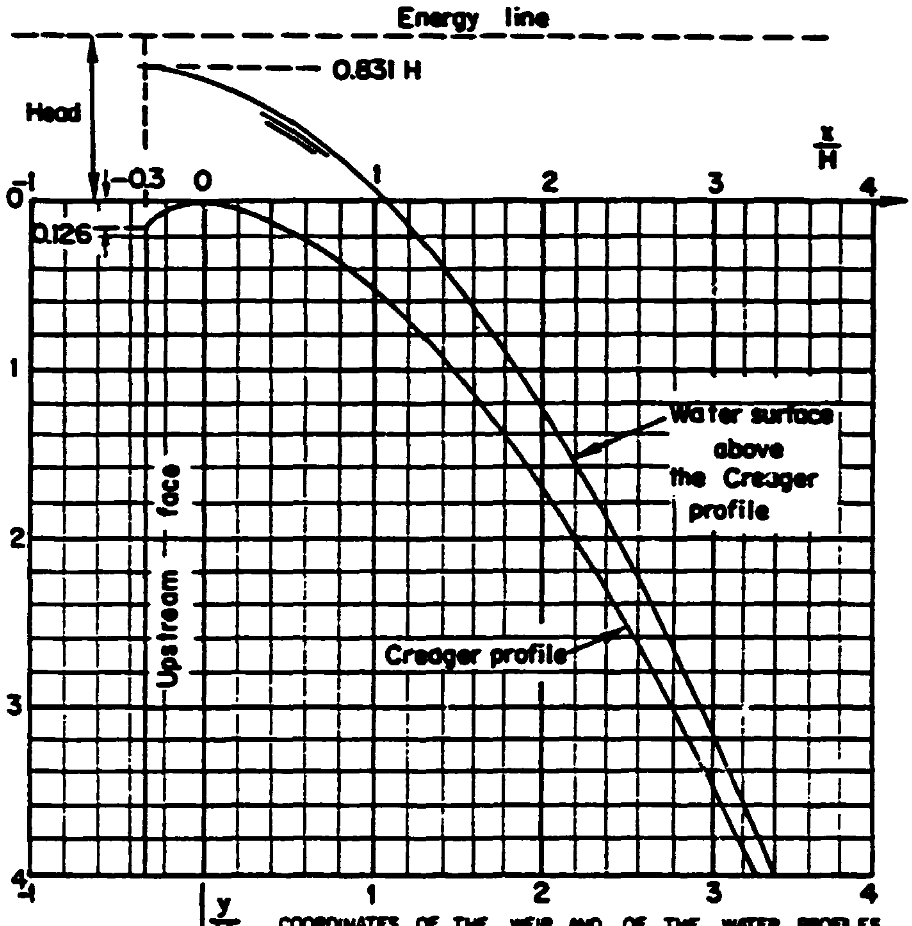
FORM	SHARP	0.6. or FLAT
μ	0.60	0.65 - 0.75
C	1.80	1.92 - 2.20

6.2.3.2 Design form of the Spillway or weir

The form of the spillway or the weir should be designed in such a way as to obtain the optimum streamflow conditions as well as an optimum pressure distribution along the spillway.

Based on the experience, a standard form for small weirs has been developed which can be applied on most cases see fig Nr. 6.10.

Were H_d is the design head (Energy head above the spillway).



COORDINATES OF THE WEIR AND OF THE WATER PROFLES

x/H	y/H	y_s/H
-0.300	0.126	-0.831
-0.200	0.036	
0.100	0.007	
0 000	0.000	
0.100	0.007	
0.300	0.060	
0.500	0.142	-0.520
0 700	0 257	-0 380

x/H	y/H	y_s/H
0.900	0.397	-0.220
1.100	0.565	-0.080
1.400	0.870	+0.280
1.700	1.220	+0.660
2.200	1.960	+1.500
2.700	2.820	+2.480
3.200	3.820	+3.500

FIG. 6.10 Design coefficients for a small weir or spillway.

6.2.3.3 Forces acting on the structure

Whilst designing the weir, the small gravity dam(if necessary) and other intake structures, it is necessary to determine the forces which may be expected to affect the stability of those structures.

The main forces which must be taken into consideration for the above mentioned structures and within the ranges of magnitudes previously selected in this report are:

- The external water pressures
- The internal (or uplift) water pressures
- Sediment pressures (if any)
- Ice pressure (if any)
- Own weight of the structures
- Additional earthquake pressures /if any).

With overflow in ungated weirs the total horizontal water pressure on the upstream face of the weir is given by the trapezoid shown on fig. Nr.6.1 in which the unit pressures at the top and at the bottom are γh_1 and γh_2 respectively in which γ is the unit weight of water assumed as 1000 kg/m^3.

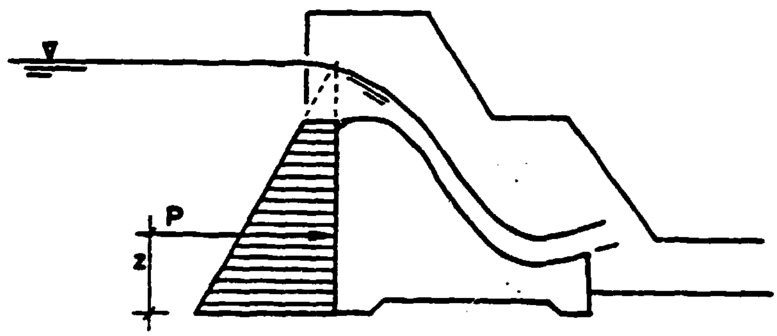


FIG. 6.1 Water pressures in an ungated weir

With overflow in gated weirs, the acting surface elevation should be increased proportionately to the gate height, increasing the total pressure P correspondingly.

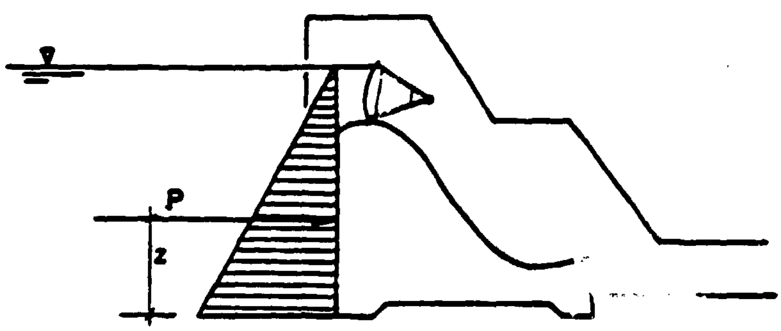


FIG: 6.2 Water pressures in a gated weir

6.2.3.4 Requirements for stability

All the gravity structures should be designed to safely withstand the following potential failure cases:

- Overturning
- Sliding or base shear
- Overstressing
- Internal (or uplift) water pressures

The potential failure due overturning of a weir or a dam occurs when the resultant force of all horizontal and vertical acting forces on the structure pass outside of the central third of the base width. However because the potential uplift pressures due to seepage are sometimes difficult to evaluate, it is necessary to check whether the vertical stress at the upstream edge of any horizontal section computed without uplift exceeds the uplift pressure at that point. If this were the case, the weir or the structure being evaluated is considered to be safe against overturning.

If the uplift pressure at the upstream face exceeds the vertical stress at any horizontal section computed without uplift, the uplift forces along an assumed horizontal fissure will increment the overturning forces on the downstream face of the weir or the dam. The potential failure due to sliding or base shear of a weir or a dam occurs when the sum of all horizontal forces (ΣV) becomes equal or greater than the shearing and frictional resisting forces between the base concrete of the structure and the foundation soil or rock.

In medium large and in large structures the criteria of the shear friction factor is used which usually requires an expensive investigation made by specialized personal.

For small structures, and specially those structures to be designed and built in developing countries, (where it is not economical or technically possible to perform tests as mentioned above, or to obtain easily reliable technical advice), the Bureau of Reclamation of the United States of America, recommends to check the structures against horizontal displacements by the method of determination of a sliding factor.

They define the allowable sliding factor as the coefficient of static friction between two sliding surfaces, reduced by an appropriate factor of safety. If f represents the allowable sliding factor, a weir or a small gravity dam or any structure is considered safe against sliding when:

$$f = \frac{\Sigma V}{\Sigma w - U} \tag{6.5}$$

The Bureau of Reclamation in its excellent Book "Design of Small Dams" give the following allowable sliding factors between concrete and various foundation materials, which being on the safe side, can be used when Laboratory test cannot be made:

TABLE Nr. 6.2
SLIDING FACTORS FOR DIFFERENT SOIL MATERIALS

MATERIAL	f
Sound rock, clean and irregular surfaces	0.80
Gravel and coarse sand	0 40
Shale	0.30

Ref.: Bureau of Reclamation "Design of Small Dams"

It is pertinent to mention that the geological investigations should determine the possibility of existence of any stratum of weak soil below the surface. In such cases, the potential sliding failure of this stratum has to be checked, including the weight of the overlying strata and the shearing resistance of the material on the weak strata.

The potential failure due to overstressing the construction materials used for the structures or the foundation soil, has to be checked and made sure that the corresponding stresses will be maintained within allowable limits.

A factor of safety equal or greater than 4 is normally recommended for concrete and masonry materials.

The foundation material in medium and large size structures is to be investigated in situ by experienced soils engineers and enough samples are to be taken to be thouroghly analysed in laboratories. However, in the case of small structures this type of procedure being not economical or technically realizable it is recommended to use the allowed bearing pressures given in the local engineering codes.

The following table gives allowable bearing values for footings of intake structures like weirs, small gravity dams, etc. The indicated figures can be use as a guide for design and control only and are not intended to replace the sound advice of local experienced engineers. If doubts remain as to the proper type of foundation materials, laboratory tests should be ordered and a soil specialist consulted.

The suggested allowable soil bearing pressure values for footings and foundations of the weir and other intake structures are as follows:

TABLE Nr. 6.3
ALLOWABLE BEARING PRESSURES FOR DIFFERENT TYPES OF SOILS

MATERIAL	ALLOWABLE BEARING PRESSURE (kg/cm2)
Sound massive igneous metamorphic or sedimentary hard rocks	110
Sound hard slate or laminated rocks	40
Residual deposits of bed rock	10
Gravel	4
Cohesionless medium dense sand	1
Cohesionless dense sand	2
Saturated cohesive sands, stiff silts and clays	0.5

6.2.3.5. The internal (and uplift) water pressures

Uplift forces occur as internal pore water pressures, on cracks, fissures and on the structure foundation. These water pressures on the pores and on fissures will act in all directions and can sometimes reduce considerably the resisting forces. Uplift pressure therefore should be included in any stability analysis of the structures.

The internal water pressures should be considered differently if the foundation of the structures are supported on a solid rock or on a pervious foundation material.

Internal water pressures on rock foundations, are assumed to be effective over the entire base of the section. It is usually assumed that the intensity of the uplift pressure at the upstream face of the structures is equal to the full water head; at the downstream face it equals zero if no tailwater is present; a linear distribution exists between the upstream and the downstream face. The proper placement of drains on the structures and drilling drainage holes into the rock foundation can help to reduce the uplift pressures.

Internal water pressures or uplift pressures under concrete structures built on pervious foundations are related to seepage flow through the pervious materials. In these cases, additional constructions are required, like cutoff walls, aprons, etc., in order to provide enough safety against potential seepage and piping occurrences.

Concrete gravity structures with more than 7 m of height on pervious foundations require detailed and extensive soil mechanics investigations. Such cases are beyond the scope of this report and should be treated by a specialist.

Structures of less than 7 m of height, resting on pervious soils can be efficiently sealed off with cutoff walls. Cutoff walls can be constructed with sheet piling of interlocking steel sections (in some countries an economical solution) with easy connection to the concrete structures; in other countries where steel is difficult to obtain, concrete cutoffs may be used under the weir section. They can be built by trenching (with machines or by hand) and forming the concrete wall into the excavated trench.

It is recommended to back fill the trench around the wall with an impervious well-compacted material. If the trench is hand made, provision should be made to protect the works with lateral trench sides supports during excavation and construction , thus preventing any serious accidents due to suddent collapse of the sides of the trench.

Cutoff walls should be always constructed on the upstream of the foundation and never on the downstream part.

A quick way to check the safety of small structures (less than 7 m height) against uplift and seepage, is to use the Lane's weighted creep theory. It is an empirical method based on a statistical experience made on many structures; it is considered to be safe enough for the design of small structures on pervious soils subjected to potential uplift pressures and seepage flow.

The main five points of Lane's conclusions are:

(1) The weighted-creep distance of a cross section of a weir or a dam is the sum of the vertical creep distances (steeper than 45°) plus one third of the sum of the horizontal creep distances (less than 45°).

(2) The weighted-creep distance head ratio is the weighted-creep distance divided by the effective head.

(3) Reverse filter drains and pipe drains are recommended aids to security from under seepage. There can reduce the weight creep ratios as much as by 10% if they are properly installed and used.

(4) Cutoffs, if any,should be properly tied in at the flanks so that water will not by-pass them.

(5) The upward pressure to be used in design may be estimated by assuming that the drop in pressure from head water to tail water along the contact line of the weir or a dam on its foundationn is proportional to the weighted-creep distance

Depending on the foundation materials Lane recommended the following weighted-creep ratios:

TABLE Nr. 6.4
WEIGHTED-CREEP RATIOS FOR DIFFERENT SOIL FOUNDATION MATERIALS

MATERIAL	RATIO
Very fine sand and silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

The weighted creep ratio should be equal or larger than the weighted length of the path divided by the water head on the structure.

As an example of the Lane's weighted theory let us assume a weir with the dimensions showed in the Fig. NR. 6.13.

By definition:

$$\textbf{Weighted creep ratio} = \frac{\textbf{Weighted length of path}}{\textbf{Water Head on Structure}}$$

Weigthed length of path = Σ Vertical path + 1/3 (Σ Horizontal path)

(see Fig. Nr 6.13)

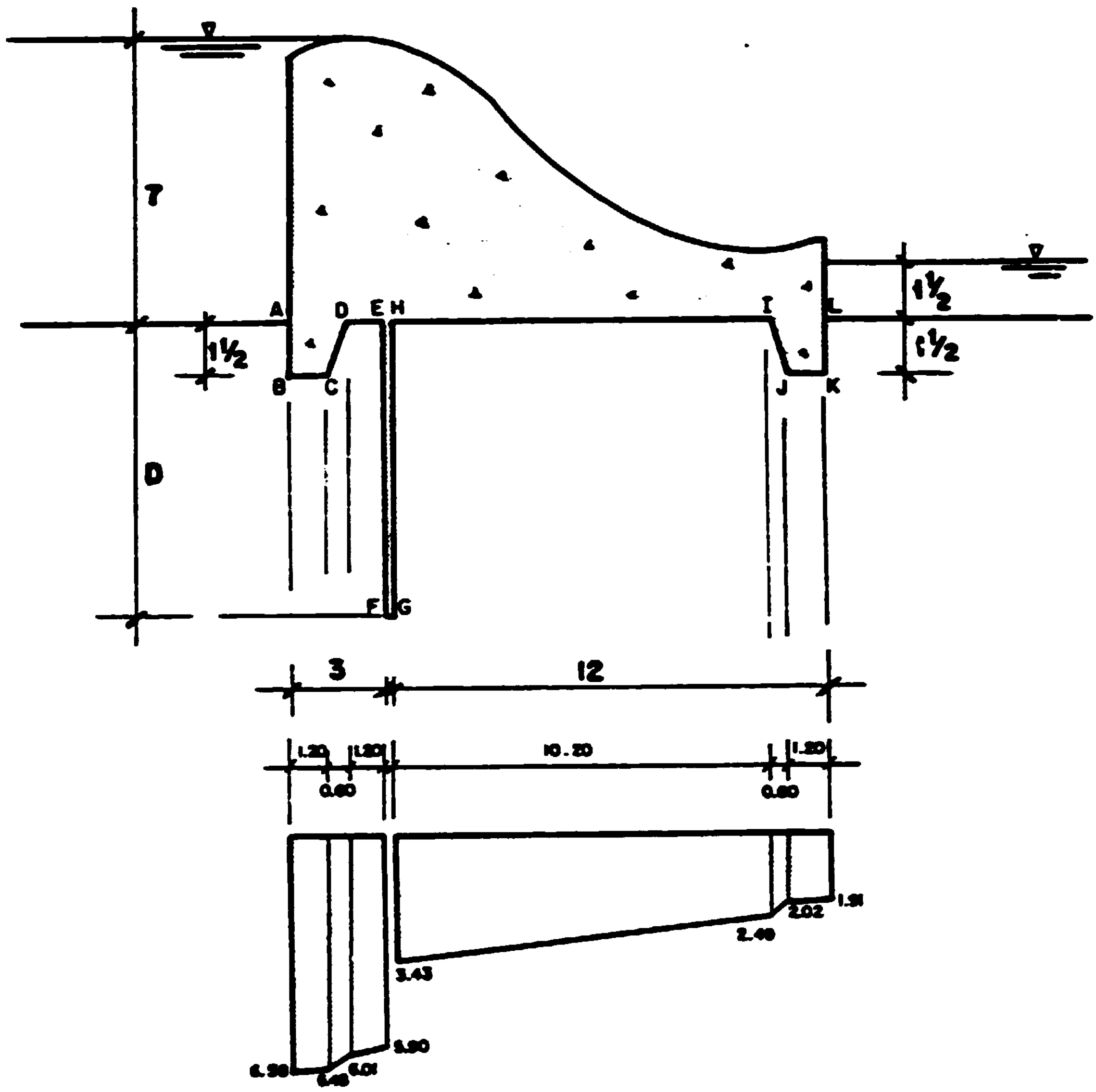


FIG. Nr. 6.13 Schematic Application of Lane's Weighted Creep Theory

Σ Vertical path = AB + CD + EF + GH + IJ + KL

Σ Vertical path = 4 (AB) + 2 (ET)

Σ Vertical path = (4 x 1.5) + 2 D = (6 + 2 D) m

Σ Horizontal path = 1/3 (BC + CE) + 1/3 (HI + JK)

Σ Horizontal path = 1/3 (3 + 12) = 1/3 (15) = 5 m

Total length of path = (6 + 2 D) + 5 m

Water Head on Structure = (7 - 1.5) = 5.5 m

$$\text{Weighted creep ratio} = \frac{(6 + 2\,D) + 5}{5.5}$$

or:

$$D = \frac{5.5\,(WCR) - 5 - 6}{2}$$

If the soil were a Medium Gravel with WCR = 3.50 (see Table Nr. 6.4)

Then: D = 4.13 m, or rounded-off 4.50 m

If the soil were a Coarse Sand with WCR = 5.00 (see Table Nr. 6.4)

then: D = 8.25 m, or rounded-off 8.50 m

According to Lane's recommended ratios (see Table Nr. 6.4) this weir would be safe on Medium Gravel with a cutoff wall of 4.50 m depth. In Coarse Sand a cutoff wall of 8.50 m depth would be required. For other materials with higher weighted creep ratios like very fine sand or silt, a much deeper cutoff wall would be required.

To visualize better the computation of the uplift pressures, let assume the case of Medium Gravel as soil foundation material with a cutoff wall of 4.50 m depth.

The uplift pressure in any point is:

$$P_x = H - \frac{L_x}{L_T} . H + H_T$$

Where:

P_x = Uplift pressure in point x
H = Water Head
H_T = Depth of Tail Water above foundation level
L_x = Length of weighted creep path of point x
L_T = Total length of weighted creep path

$$\text{Uplift at point } B = 5.5 - \frac{1.5}{20} \times 5.5 + 1.5 = 6.59 \text{ m}$$

$$\text{Uplift at point } C = 5.5 - \frac{1.5 + 1/3 \times 1.2}{20} \times 5.5 + 1.5 = 6.48 \text{ m}$$

$$\text{Uplift at point } D = 5.5 - \frac{1.5 + 1/3 \times (1.2+0.6)+1.5}{20} \times 5.5 + 1.5 = 6.01 \text{ m}$$

$$E = 5.5 - \frac{1.5 + 1/3 \times (1.2+0.6)+1/3 \times (1.2)}{20} \times 5.5 + 1.5 = 5.90 \text{ m}$$

$$H = 5.5 - \frac{1.5 + 1/3 \times (1.2+0.6)+1/3 \times (1.2) + 4.5+4.5}{20} \times 5.5 + 1.5 = 3.43 \text{ m}$$

$$I = 5.5 - \frac{1.5 + 1/3\,(1.2+0.6+1.2) + 4.5+4.5+ 1/3(10.20)}{20} \times 5.5 + 1.5 =$$

$$= 2.49 \text{ m}$$

$$J = 5.5 - \frac{1.5 + 1/3\,(1.2+0.6+1.2) + 4.5+4.5+ 1/3(10.20)+1/3(0.6)+1.5}{20}$$

$$\times 5.5 + 1.5 = 2.02 \text{ m}$$

$$K = 5.5 - \frac{1.5 + 1/3\,(3.0) + 4.5+4.5+ 1/3(10.20)+1/3(0.6)+1.5 + 1/3(1.2)}{20}$$

$$\times 5.5 + 1.5 = 1.91 \text{ m}$$

Total uplift pressure = P_T

P_T=[1/2(6.59+6.48)x 1.20+1/2 (6.48+6.01)x 0.60 + 1/2(6.01 + 5.90)x1.20

+1/2(3.43+2.49)x10.20 + 1/2(2.49+2.02)x0.60 + 1/2(2.02+1.91)x1.20]

x 1 m width x 1000 kg/m^3

P_T = 52 640 kg/1 m

or **P_T = 52.6 Ton/1 m**

(52.6 Tons of uplift pressure per linear meter of weir)

6.3 DROP INTAKES

As an alternative to conventional water intakes, drop intakes (see figure 6.14) do not require a bend of the river and a lateral entrance with vertical side track; drop intakes can literally be placed everywhere along a river or a creek, regardless whether the river has a bend or not. Suitable sites as for instance narrow passes with sound foundation rock or reliable foundation materials on which to place the intake structure represent convenient and economical solutions. It is evident that drop intakes are particularly well suited for locations in steep terrains.

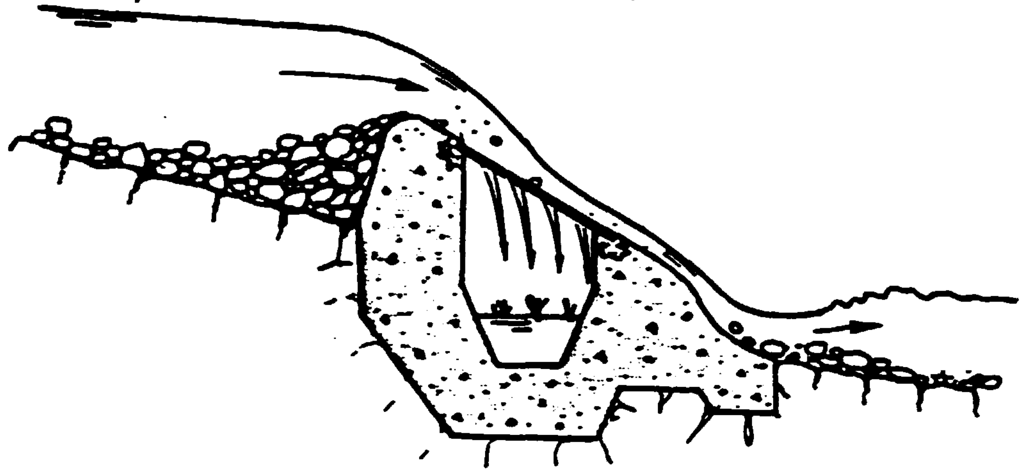


FIG. Nr. 6.14 Schematic section of a drop intake

6.3.1. Design of a drop intake

It is a known fact that in mountainous zones, even small catchment areas can produce floods with much higher specific discharges per square kilometer than medium or large size flat areas. Most of the drop intakes expect therefore a considerable larger amount of sediments than the run-off-river type intakes.

Drop intakes are build usually in areas of difficult access, but this apparent disadvantage is compensated by the fact that their require very little maintenance.

In this report the design criteria adopted for drop intakes, deviates very little from the standard designs made by the TIWAG in Austria or the EDF in France. The only difference is the slope angle of the entrance rack. For small discharges ($Q < 3\ m^3/s$) the writers recommend , to use a slope for the entrance rack of 70% insted of the 80% used for larger discharges. This value Of 70% is based on the experience gained in other similar intakes like Otema, Brenney, Leteygeon, Stechelberg, etc.

The recommended space between rack bars is 25 mm for discharges of $Q < 3.5\ m^3/s$ (see Fig. Nr. 6.15).

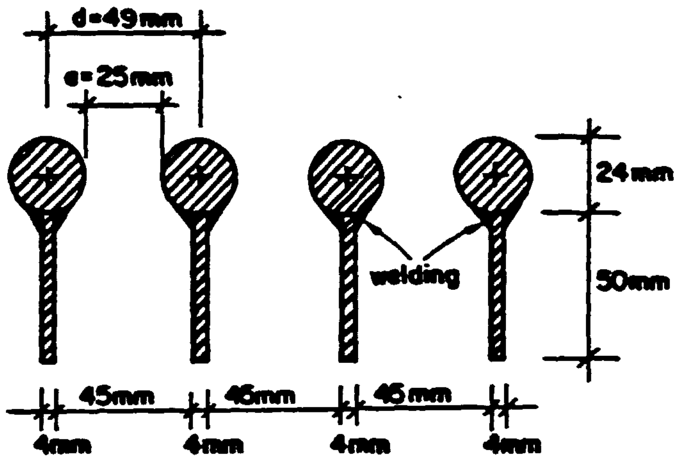


FIG. Nr. 6.15 Typical rack for a drop intake with $Q < 3.5\ m^3/s$

With this design, most of the alluvium material with diameters above 20 mm will pass over the rack.

Material with diameters between 0.5 and 20 mm will therefore enter the desilting basin. The desilting basin for this type of intake is in the average 10% larger than a desilting basin for the same Q_{design} but for a run-off-river intake with a vertical side rack, because in this case small gravel will enter the basin and will deposit at the entry zone of the basin (see fig. 7.2.)

The main components of a drop intake are the following :

◇ A forebay excavated generally in rock to produce a more regular and quiet approach of the water flow on the rack.

◇ Two openings with slots which allow to close them with wooden stop logs, on the right side of the intake; the logs can easily be taken out if necessary for cleaning the forebay or the rack.

◇ An intake opening of width B with stops logs permitting the closure of the intake with two or three elements with a maximum height of 0.40 m each.

◇ An device for the erction of the stop logs.

◇ A service footbridge

◇ Stairs on both sides of each opening.

◇ A trash rack with a 70% slope (for $Q_{design} < 3.5\ m^3/s$)

The intake must be checked at least once a week and the rack cleaned from leaves and gravel; further the thickness of the sand-gravel deposit should be checked at least at three points in the desilting basin. If necessary the flush gate should be opened.

The alluvium, tree branches and stones should be removed after ocurrence of a flood. Any minor repairs should be made at the same time.

For large flood (1000 year), the water will pass at a maximum of 0.70 - 0.80 m above the well crest, for drop intakes with a $Q_{design} < 3.5\ m^3/s$

If the river or creek is known as having occurrences of flash floods, it is recommended to :

- Place the intake on the river side.
- Prepare a special upstream protection wall.
- Adapt the design of the intake for special features.

6.3.2. Dimension of the rack

The dimension of the rack can be estimated as follows :

At the entrance of the rack the critical depth (see fig. 6.16) can be calculated from the following formula:

$$h_c = \sqrt[3]{(Q^2/B^2\ g)} \qquad (6.6)$$

For a $Q < 3.5\ m^3/s$ and an angle $\beta = 35°$ the depth of water h (see fig 6.16) is:

$$h = 3/4\ .\ h_c \qquad (6.7)$$

The discharge capacity is estimated with the formula:

$$Q = 2/3\ .\ C\ .\ \mu\ .\ B\ .\ L\ .\ \sqrt{2gh} \qquad (6.8)$$

where:

h = water depth at the beginning of the rack = $3/4\ .\ h_c$
h_c = critical depth
$c = 0.6\ .\ (a/d)\ .\ (\cos \beta)^{3/2} = 0.6\ .\ (25/29).(\cos 35°)^{3/2} = 0.72$
μ = Rack coefficient = 0.90
B = Rack width
L = Rack length
β = angle of the rack with the horizontal
a = opening between rack bars
b = distance between bars centers

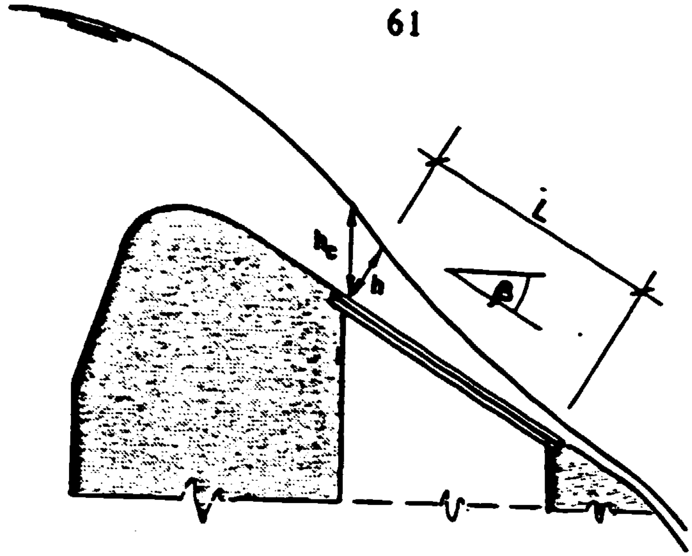


FIG. Nr. 6.16 **Depth and lenght of the rack of a drop intake**

For drop intakes with a $Q < 3.5\ m^3/s$ the formula for Q can be simplified:

$$Q = (0.66).(0.72).(0.90).(4.43).B.L.\sqrt{h}$$

or

$$\mathbf{Q = 1.90\ B.L.\sqrt{h}} \qquad (6.9)$$

Practical experience shows that :

$$\mathbf{B\ approx. = 0.60 . L} \qquad (6.10)$$

For proper working conditions B should not be greater than 1.8 to 2.00 m. When Q requires a width B greater than 2.0 m it is advisable to design two parallel intakes instead of increasing the width of a single one.

The length L is given a factor of safety of 1.3 at least (because the possibility of a partial obstruction of the rack, a possibility of negligence in the maintenance, and to have a reserve flow during flushing operations).

For length of racks greater than 2.0 m it is recommended to reinforce the rack with an intermediate support to avoid excesive bending moments of the bars, or undesired vibrations.

6.3.3. <u>Operation of Drop Intakes</u>

A drop intake function practically without any operating instructions. An instruction manual for the flushing operation of the desilting basin is however required.

After the water has fallen below the rack, it passes through the entrance canal and through a control section, which limit the discharge flow during floods, in a way that not more than 30% of the flood discharge can enter the desilting basin, the rest water being diverted through a spillway down stream of the intake structure.

A flood protection wall should be placed at the inlet of the canal.

Any excess of water passing the desilting basin, can be deviated at the spillway of the forebay or the entrance of the penstock.

CHAPTER 7

7. DESILTING BASIN

7.1 Generalities

The conventional water intakes are designed in a way that allows to flush the bottom sediments as easily as possible. Sediments cleaning problems vary from one river to another; some rivers bring clean water, while others with the same discharge are loaded with sediments. Whilst gravel and rolling stones are deviated and passed through the sluice channel, there is the possibility that some entrainment of sand and other fine granular material remains.

Sand passing through the turbines, has usually catastrophic consequences, eroding the impellers and casings in a short time.

To avoid this danger, it is therefore imperative to install a desilting basin.

It may be mentioned here that there are many systems of desilting, however, for the purpose of this report, the writers have decided to select only two systems, one for a run-of-river plants, and one for drop intakes, selecting of course, desilting plant types that have shown to have a maximum of advantages and a minimum of disadvantages in the range of discharges be[illegible] considered.

The first desilting type is the one that correspond to the run-of-river intakes, see chapter 6 numeral 6.2 of this report.

The second one corresponds to desilting basins for drop intakes which operate in a different way, because a larger quantity of sediments and sand is picked up in the intake; consequently a different approach for its design is required (see chapter 6 numeral 6.3).

For intakes with medium and large discharges (> 3.50 m^3/s), mechanized desilting systems are usually used; they are beyond the scope of this report.

For mini and micro power plants, the desilting works have to be of a very economic design, and mechanized systems are not justified economically. However, it is well known, that in larger schemes, mechanized systems of desilting are not only economic but necessary as well. A well known system is the Bieri system. The systems proposed in this report can be reälized exclusively with local personnel and local resources.

7.2 Conventional Desilting Basins for Run - of - river Intakes

In conventional run-of-river intakes, the larger part of coarse sediments and gravel is separated from the main stream directly at the entrance by means of an entrance sill, diverting the sediments directly into the sluice channel, where they can be flushed down stream periodically.

The trash rack is designed to provide a maximum of protection against sediments, however, a small percentage of sediments carried by the river always manage to get into the entrance of the water ways, specially during high water periods.

If the hydrologic study shows that granular material with grain diameters of more than 2 mm can pass the trash rack, a desilting plant with its corresponding basin should be constructed. This is all the more important if the turbines are of the Francis or the Pelton type; for Crossflow turbines this problem is somehow less critical.

In order to be able to provide a longer life to the turbines and to the overall scheme in general, sediments have to be removed from the system.

The conventional examples of desilting basins presented in this report have been elaborated according to the experience gained in other similar projects.

Our considerations assume that, the sediments usually present have a grain diameter within a range between 0.2 and 0.5 mm.

$$\mathbf{0.2\ mm < diameter < 0.5\ mm}$$

According to Huber, the critical water velocity can be estimated using the following formula :

$$V_{cr} = 0.44 \times \sqrt{d} \qquad (7.1)$$

which means that water velocities of the order of 0.20 m/s will remove all particles with diameters greater than 0.20 mm.

This limit permit to calculate the area of trash rack required for a discharge Q_{design}, and to calculate the main dimensions of the desilting chamber.

The following Table 7.1 gives approximately the main dimensions of the desilting basin for conventional run-of-river intakes as a function of the design discharge:

TABLE Nr. 7.1
MAIN DIMENSIONS OF A CONVENTIONAL DESILTING PLANT

Q(m3/s)	B(m)	L(m)	D(m)	W(m)
1.5	2.60	21.00	2.25	0.90
2.6	3.50	28.00	3.10	1.10
3.5	4.00	32.00	3.65	1.30

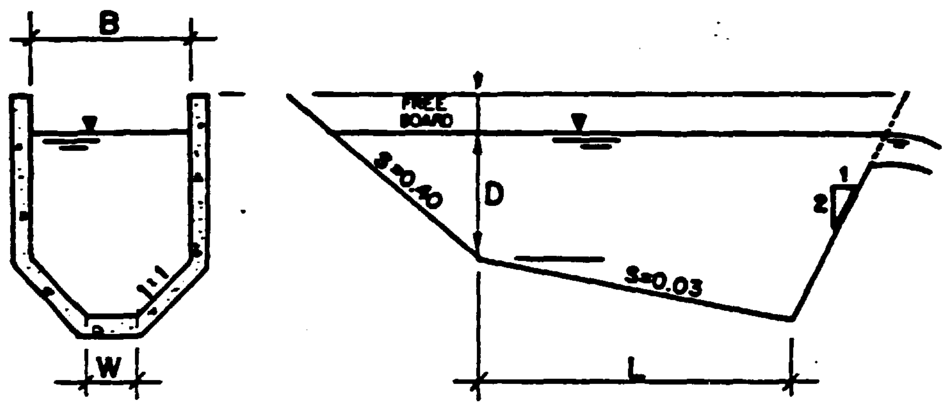


FIG. Nr. 7.1 SCHEMATIC DIMENSIONING OF A CONVENTIONAL DESILTING BASIN.

For water discharges of more than 4 m^3/s it is recommended to install two or more smaller units for desilting instead of a larger one, thus limiting the water discharges to not more than 4 m^3/s per unit.

This system requires that the depth of coarse sediments in front of the intake sill and sluice channel should be checked periodically and regularly, to avoid an excessive accumulation of sediments

For depths of sediments of 0.5 - 0.7 m the flush gate should be lifted and the basin should be continuously flushed until the entrance is again clean of sediments.

7.3 DESILTING BASINS FOR DROP INTAKES

In the main entrance drop intakes only the coarse gravel and stones from the water are separated as they pass on top of an inclined rack (with a slope of about 70%)

The separation of gravel requires an extra lenght of the inlet in the basin equal to about three times the width of the basin ; the basin has therefore to be designed differently as compared to the type with vertical side entry rack.

The bottom slope is made in two parts, the first with a length L1 and a bottom slope of S = 0.06 , and the second with a length of L_2 = 3B with a bottom slope of S = 0.10. This discontinuity of the bottom floor slope, permits an easier and more eff'cient cleaning of the basin, as a supercritical flow during flushing is created. The sediments accumulate on the bottom as depicted in figure 7.2

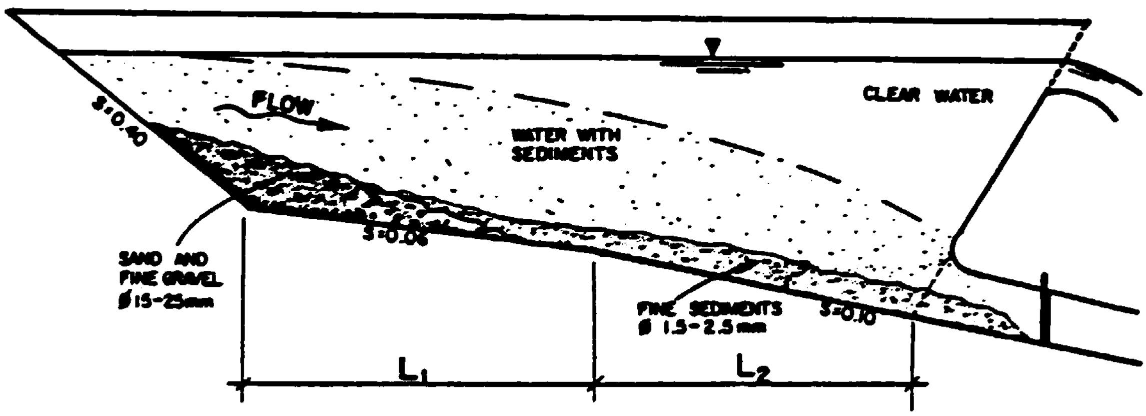


FIG. Nr. 7.2 Schematic description of the sedimentation process in the desilting basin

The following restrictions apply to this type of system :

- During flushig operation, the power plant should not be in operation
- The depth of sediments deposit should be regularly checked
- The outlet of the flush channel has to be placed high enough above the river bed.
- The regulation is made with the turbine and a level control in the forebay.

- The intake and the desilting operation normally do not require a special control.

The main dimensions of desilting basin for drop intakes as a function of the design discharge Q_{design} are as follows:

TABLE Nr. 7.2 MAIN DIMENSIONS OF DESILTING BASINS FOR DROP INTAKES

Q(m3/s)	B(m)	L1(m)	L2(m)	L(m)	D(m)	W(m)
1.10	2.20	15.40	6.60	22.00	2.10	0.70
1.50	2.60	16.20	7.80	24.00	2.25	0.90
2.50	3.40	20.80	10.20	31.00	3.00	1.10

NOTE: FOR VALUES OF Q GREATER THAN 3.0 m^3/s THE DESIGN HAS TO BE TAYLORED TO EACH PARTICULAR CASE.

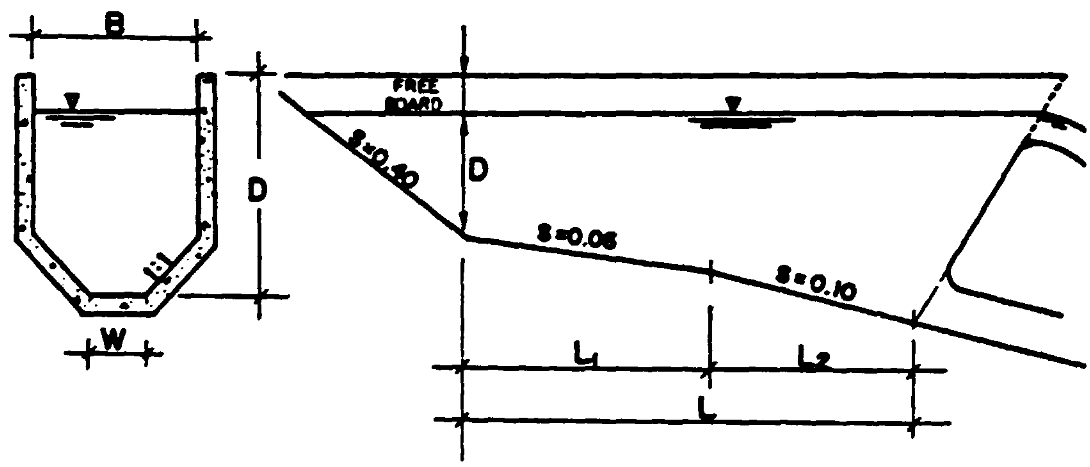


FIG. Nr. 7.3 SCHEMATIC DIMENSIONING OF A DESILTING BASIN FOR A DROP INTAKE

CHAPTER 8

8. ADDUCTION CANAL

It is important to control the amount of water that can enter the adduction canal in order to avoid undesirable spilling of water from the canal sides.

At the entrance of the canal a side spillway should be planned and constructed as a protection of the canal against floods. A frontal beam placed on top of the canal entrance will divert any excess of water during a flood to the lateral spillway .

It is a world wide common practice to close the intake entrance during very short periods when very high floods are expected, in order to protect the canal , and all the other structures, such as the penstock, the power plant with turbines, gates, etc. For small and mini power plants it is more favourable and cheaper to close the plant for a short period of time, than to have to repair it after the flood.

A canal has to be adapted to the natural configuration of the slopes and the topographic contours.

For the range of water discharges considered in this report, the canal slope recommended from the experience gained with other schemes is of S = 0.0012 with a bottom of concrete type PC - 300 of 0.30 cm thickness with a light steel bars reinforcement. The sides are usually made of masonry with 5 : 1 slopes, or of concrete with vertical walls.

For very small plants, for which concrete or masonry is relatively expensive, the canal may be cut directly into the ground. This is generally not recommendable but should it be made this way, care should be taken not to exceed the permissible erosion velocities; otherwise damaged turbines and equipments may result thereof.

In the technical literature it is recommended not to exceed a water velocity of 0.75 m/s in canals without lining, however, the following table 8.1 gives water velocities limits in canals build directly on various soil materials :

TABLE Nr. 8.1
ALLOWABLE LIMIT OF WATER VELOCITIES IN CANALS AGAINST EROSION.

SOIL MATERIAL	LIMIT OF WATER VELOCITY (m/s)
Fine Sand	0.40
Sandy Clay	0.50
Clay, alluvial lime without cohesion	0.60
Ordinary clay, fine gravel	0.70
Alluvial lime with some cohesion or a mixture of gravel sand and clay	1.00
Gravel (Ø < 1 cm)	1.20
Gravel (1. cm < Ø < 5 cm)	1.50
Schists	1.80
Stratified rock	2.40
Hard rock	4.00
Concrete	4.50

Should the water flow with too low velocity, sediments begin to settle and they eventually obstruct the canal. For that reason is recommended to choose a design with water velocities that are somewhat lower than the allowable water velocities against erosion; a factor of safety of the order of three should be the rule, but on the other hand, the same water velocities should be keep high enough to avoid a sedimentation on the adduction canal.

Kennedy gives an empirical formula to estimate the sedimentation velocity in a canal as a function of the water depth and of a constant:

$$V_s = C \cdot t^{0.64} \qquad (8.1)$$

In which:

V_s = sedimentation velocity

C = a constant equal to 0.548 in alluvial soils and $C \geq 0.437$ in other soils

t = water depth in canal (m)

In the abscence of better values Table 8.2 gives as a reference some limit velocities of sedimentation:

TABLE 8.2
SEDIMENTATION VELOCITIES

MATERIAL	LIMIT VELOCITY (m/s)
Clay	0.10
Fine Sand (ø = 0.002 m)	0.15
Sand (ø = 0.005 m)	0.20
Fine Gravel (ø = 0.008 m)	0.30
Gravel (ø = 0.025 m)	0.60

The usually used canal sections for small hydropower schemes have the following sections (see FIG. Nr. 8.1) the main dimensions of which are given in Table 8.3 in function of the design discharge:

TABLE 8.3
MAIN DIMENSIONS OF CANALS

Q	V	B_1	B_2	W	H	RADIUS IN CURVES(m)
m^3/s	m/s	m	m	m	m	m
1.00	1.00	1.35	1.80	0.70	1.15	8 - 10
2.60	1.20	1.85	2.45	1.05	1.55	10 - 12
3.50	1.30	2.10	2.75	1.15	1.65	12 - 15

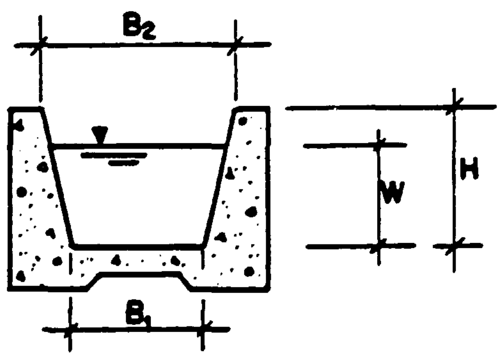


FIG. 8.1 Typical canal sections for small hydropower projects

The water velocities are taken with a factor of safety of the order of 3 to 4 against erosion, but are at the same time large enough to prevent sedimentation in the bottom of the canal.

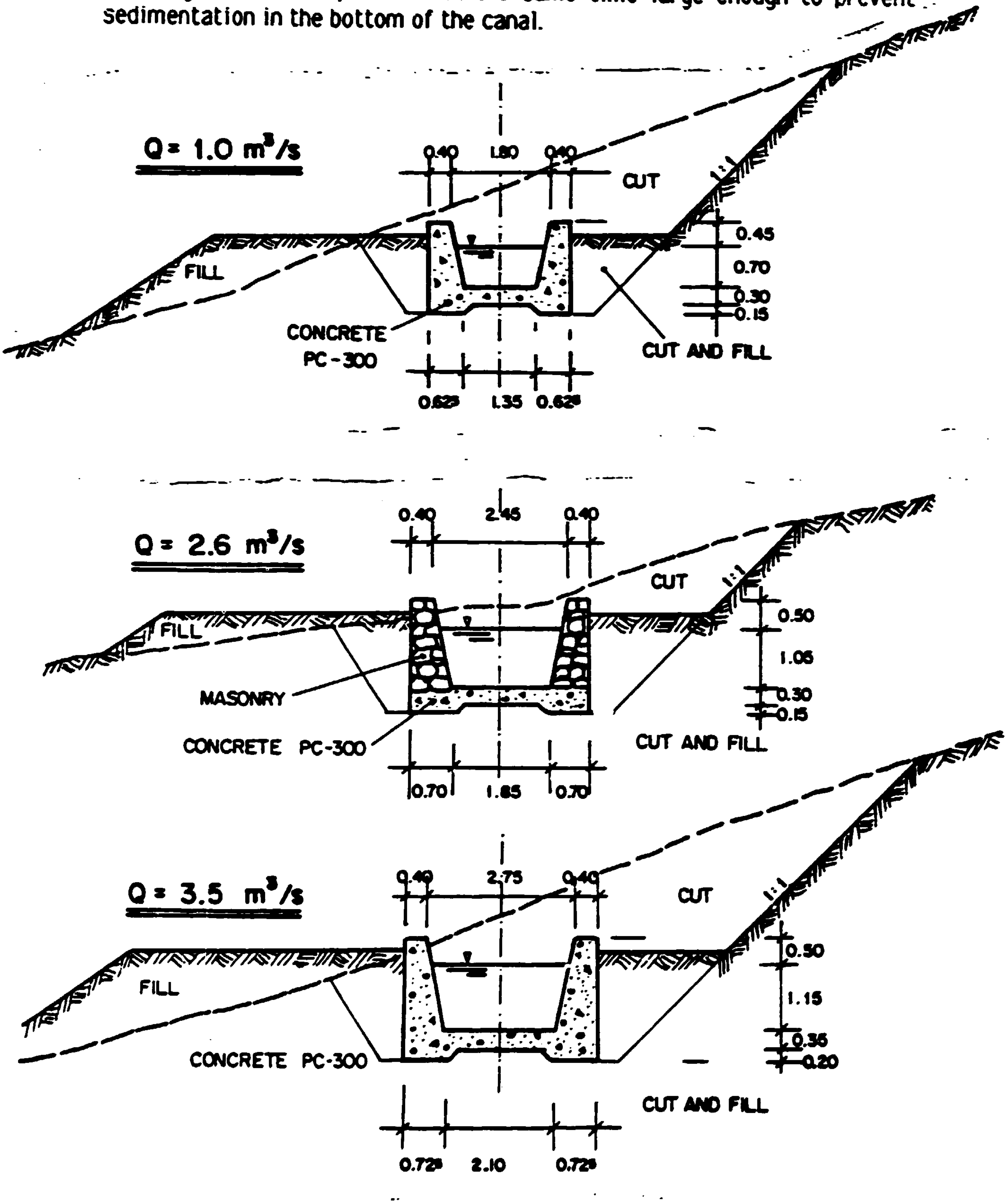


FIG. 8.2 <u>Examples of Adduction Canals with slope S = 0.0012</u>

CHAPTER 9

9. FOREBAY

It is important to avoid the entrance of air into the penstock and from there into the turbines.

The construction of a forebay at the entrance of the penstock permit not only to guarantee the conduction of water free of air, but allows to maintain a relatively constant head of water into the system. The design range varies from a simple pond to very sophisticated structures. Its size varies, depending on the maximum supply required for energy generation. However, if storage is desired, its size, will be dictated by the required volume storage. It is evident that a forebay should have a minimum cost but its size should be large enough to fulfill its purpose.

High pressure systems require usually a reserve volume that is of the order of 2.3 to 3 times larger than the forebays required for systems for low pressures.

For small plants, the surface area of forebays required can be estimated as a function of the maximum discharge with the following rule of thumb:

TABLE 9.1
RECOMENDED FOREBAY AREA FOR SMALL HYDROPOWER PLANTS

Discharge Q(m^3/sec)	0 - 5	5 -25	25 - 50
Area of low pressure forebay A (m^2)	60 - 150	150 - 400	400 - 500
Area of a high pressure forebay A (m^2)	200 - 400	400 - 900	900 - 1200

NOTE: The forebay areas above mentioned are only valid for the design examples presented in this report.

If the forebay is intended to be used as a desilting basin, the design has to be properly adapted, in order to avoid that debris, and sediments pass through and into the turbines. From the writers' experience, it is not recommended to use the forebay as a settling basin or desilting works; it is preferable to construct a separate structure for that purpose.

The forebay should provide a storage of water large enough to supply the turbines with adequate water quantities as required for starting and for regulation. The streambed can sometimes be transformed into a small storage; in this case, the forebay will be designed as small as possible. Where this is not the case, the forebay will be increased in volume. An economical study will however be required to make sure that the cost benefit ratio of the forebay is positive

The minimum storage required by a forebay is dictated by the water demands originated by a sudden increase in the loading on the turbines, which is a transient condition.

Forebays should be provided with an emergency spillway to be able to cope with occasionally occurring high inflow of water into the forebay which exceed the outflow through the penstock. The spillway will also function in the case of a black-out or a power failure which requires the gates to be closed.

There are many types of forebays, however, the forebays presented as prototypes in this report have been selected that present most benefits and positive performances.

CHAPTER 10

10. PENSTOCK

10.1 Generalities

The penstock is an essential part of any hydropower scheme.It conveys the water by the shortest way from the forebay into the turbines; it should therefore be constructed to fulfill all requirements of safety and economy

Its design should be aimed at maintaining the L/H relation as small as possible; consequently, the penstock should be as vertical as possible in relation to the horizontal elevation curves of the topographic contours. This assumes stable geological conditions, enabling the penstock to be anchored on solid rock.

If a part of the penstock is to pass on overburden, it is recommended to protect the surface with stone, (dry masonry), especially in areas with heavy rainfall, in order to prevent transversal and longitudinal ditches.

Depending on the head of the scheme, the size and the natural conditions, the material of the penstock can vary. For low and medium heads and short distances, a simple Polyvinyl Chloride pipe (PVC), or concrete pipes could be used, Steel pipes are applicable for the medium and high heads and longer distances.

The most commonly used penstock is steel pipe, which offers the the highest technical and maintenance advantages.

The penstock can be installed above ground or buried in the ground, depending on the natural soil conditions. The "above ground systems" are easy to check and to maintain, but in some countries are more susceptible to acts of sabotage , which today is an additional point to consider whilst in the design stage.

It would take considerable more time and space to present each possible combination of penstock. In this report the writers have concentrated on the main design considerations proposed. Only one example of a penstock made of steel has been proposed.

For small head, the penstock can be made of PVC. However, for most schemes, steel pipe is required and solid anchor blocks have to be provided to assure that the resisting forces will not displace and damage the rigid pipe.

Anchor blocks are therefore an important part of the penstock overall system.

When the penstock exceeds certain lengths expansion joints have to be provided to to take into account displacements in the longitudinal direction caused by temperature differences or by changes of the hydrostatic forces inside the pipe which tend to separate the pipes or the joints.

Penstocks cannot always be built in a straight line, they have to be adapted to existing topographic conditions, and therefore horizontal as well as vertical (and combined) bends will be required to adapt the penstock to those topographic features. These bends cause, strong forces to appear, forces that have to be neutralized and taken up by the anchor blocks.

10.2. Penstock location

The location and disposition of a penstock is given by the topographical and geological conditions, the location of the water intake and desilting structures and outlet works, the relative location of the power house, as well as the method used to divert the river during construction.

For power houses with two or more turbines, the use of an individual penstock for each turbine, or a simple penstock with a division system to feed all units is governed by the economics and by the necessary flexibility of operation.

In this report only the single penstock system has being considered.

The penstock should be designed to convey water to the turbine with a minimum of head loss. Usually an economic study determines the size of the penstock; but the final design has to be based with combined consideration of the technical conditions required by the engineering criteria used, and the costs and economic considerations as well.

10.3 Penstock Dimensioning

Approximate diameters based on the more economic considerations can be estimated using the following relations:

a) For H < 100 m $D = \sqrt{0.05\,Q^2}$ (10.1)

b) For H > 100 m $D = \sqrt[7]{5.2\,Q^3/H}$ (10.2)

The minimum water velocity is a function of the relation L/H and can be see in Table 10.1

Table 10.1 minimum water velocities for penstock design

Ratio L/H	Minimum Water Velocity (m/s)
1	5.70
2	4.50
3	3.20
4	2.30

Those values are taken from the experience obtained in many projects.

The maximum water velocity is as a general rule is not larger than 6 m/s.

10.4 Head losses in penstocks

The hydraulic head losses in a penstock are proportional to the length of the penstock and to the square of the water velocity.

The main head losses are as following:

- ◇ Losses at the trash rack in the forebay
- ◇ Entrance losses
- ◇ Losses due to pipe friction
- ◇ Losses due to pipe bends
- ◇ Losses due to valves and regulating organs.

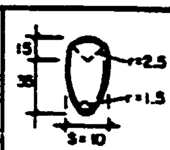
The losses at the trash rack for the range of values under study in this report are:

$$h_1 = K_1 \frac{v^2}{2g} \sin \alpha \qquad (10.3)$$

where:

$$K_1 = \beta \, (s/b)^{4/3}$$

β, s and b as depicted in Fig. 10.1

FORM					
VALUE OF β	2.40	1.80	1.04	1.07	0.76

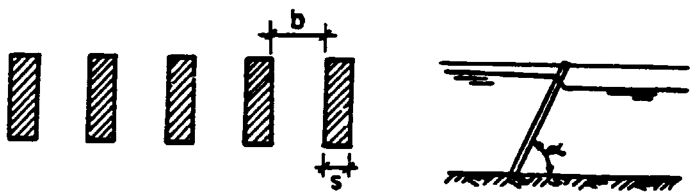


Fig. 10.1 Coefficient of losses at the Trash Rack

The **losses at the entrance** can be calculated by the following formula:

$$h_2 = K_2 \frac{v^2}{2g} \qquad (10.4)$$

where K2 can be obtained from figure 10.2

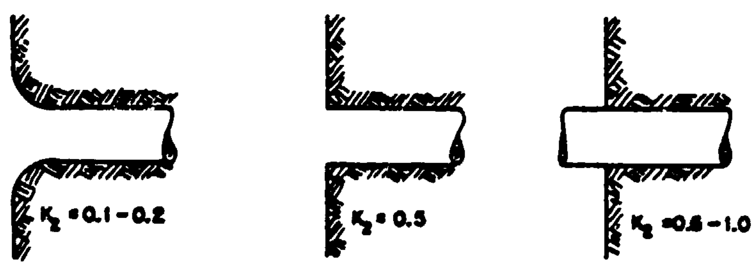


Fig. Nr 10.2 Coefficient K_2 of losses at the entrance

The **losses due to pipe friction** can be calculated by the following formula:

$$h_3 = \frac{v^2 \; L}{K^2 \; R^{4/3}} \qquad (10.5)$$

where:

K = Strickler Coefficient of Friction
K = 80 for concrete
K = 90 for steel
K = 100 for PVC
L = Length of pipe (m)
R = Hydraulic radius
v = velocity of water (m/s)

The **losses due to pipe bends** are:

$$h_4 = K_4 \frac{v^2}{2g} \qquad (10.6)$$

where K_4 can be obtain from Figure 10.3. It is a function of the bend angle, the bend radius and the pipe diameter.

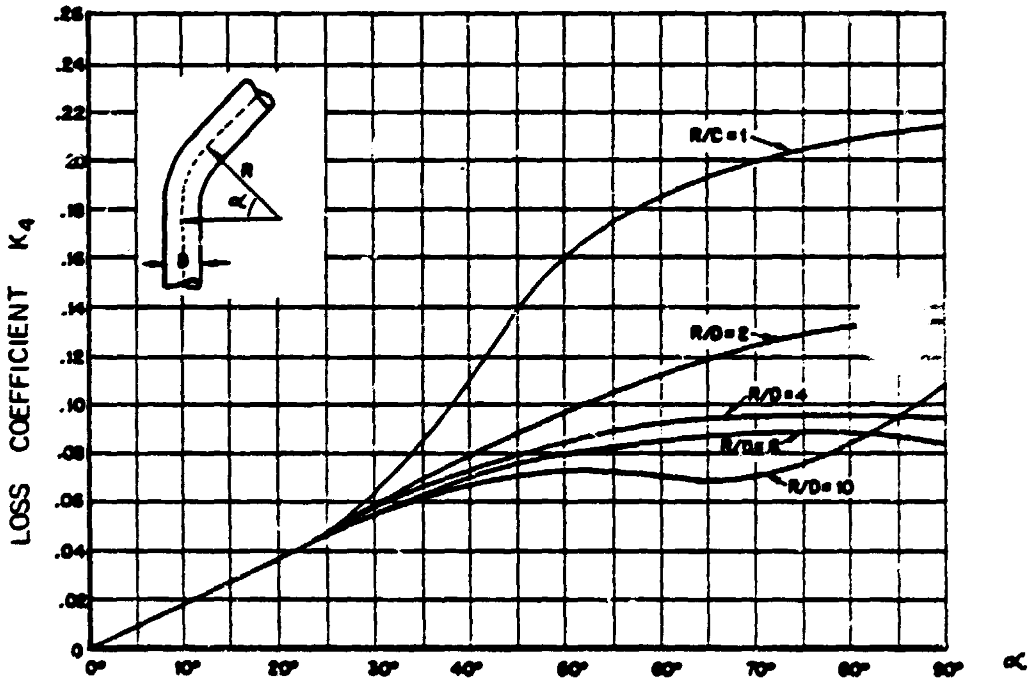


Fig. Nr 10.3 Coefficient K_4 of losses due to pipe bends

The losses at Gate valves and regulating organs are:

$$h_5 = K_5 \frac{v^2}{2g} \qquad (10.7)$$

where K_5 can be estimated from the Tables 10.2 and/or 10.3

Table Nr 10.2 Values of the Coefficient K_5 for Gate Valves

D (m)	% of Gate valve opening				
	10	25	50	75	100
0.30	56	12.0	2.50	0.50	0.07
0.50	48	10.3	2.14	0.40	0.06
1.00	40	8.57	1.79	0.34	0.05
2.00	32	6.80	1.43	0.27	0.04

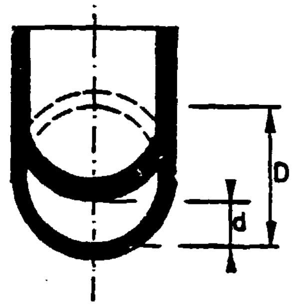
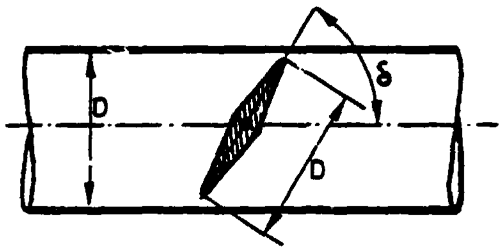


Table Nr. 10.3 Values of the Coefficient K_5 for Butterfly Valves

δ	K_5	δ	K_5	δ	K_5
0°	0.20	20°	1.54	40°	10.8
5°	0.24	25°	2.51	45°	18.7
10°	0.52	30°	3.91	50°	32.6
15°	0.90	35°	6.22	55°	58.8



10.5 Effects of Water Hammer

A rapid opening or closing of the turbine gates produce a reaction in the form of a pressure wave in the penstock, commonly known as a water hammer.

The intensity of the water hammer is proportional to the speed of propagation of the pressure wave created and to the velocity of the flow destroyed

The traveling time of the water hammer can be calculated with the expression:

$$T_s = \frac{v \; L}{g \; H} \qquad (10.8)$$

For traveling times shorter than 1.3, the volume of water contained in the pipe will be sufficient, and no surge chamber will be required.

The equation that gives the maximum increase in head for closures in a time smaller than 2L/a seconds is:

$$\Delta H = \frac{a\,v}{g} \qquad (10.9)$$

where:

ΔH = Maximum head increase (m)
a = velocity of the pressure wave (m/s)
v = velocity of flow (before destroyed) m/s
g = acceleration of gravity = 9.81 m/s^2

As water hammer surges occur under emergency conditions that could jeopardize the safety of the structure if they are not considered during the design phase, their magnitude should be estimated and the pipe thickness calculated and designed to take care of the resultant total head.

Usually surge tanks are connected to the penstock to reduce the water hammer effects.

In this report the water hammer effects as well as the design of surge chambers have not been included because in small and mini hydropower schemes they are in most cases not required, particularly if the penstock diameter is large enough and the length of the penstock relatively short. However, its necessity should be checked for each project.

10.6 Pipe Shell

The penstock should be designed to be able to resist the total head comprising the static and the dynamic heads (incl. water hammer).

Allowance should be made for temperature changes and beam stresses, in addition to the internal stresses.

The hoop or tangential tension in a thin pipe due to internal pressure is:

$$S = \frac{D\ p}{2\,t\,e} \qquad (10.10)$$

where:

S = Hoop tension (kg/cm^2)
D = Inside diameter (cm)
p = internal pressure (kg/cm^2)
t = plate thickness (cm)

e = efficiency of joint
ASTM estipulates:70-100% for double welded and 65-90 % for single welded.

Independent of the pressure, the minimum thickness of steel for a given penstock diameter can be computed with the following formula:

$$t_{min} = \frac{D(mm) + 130}{10\ 000} \qquad (10.11)$$

where: D = inside diameter in mm

10.7 Expansion Joints

Penstocks laid underground are affected by the temperature of the conveyed water and the temperature of the surrounding soil.

Penstocks constructed above the ground are affected by the temperature of the conveyed water and the temperature of the air surrounding the pipe.

An average of length of steel pipe (disregarding the frictional resistance) increases by 0.000 007 L per degree centigrade of temperature change

One of the most effective expansion joints is the sleeve type, in which the longitudinal movements are enabled by two fitting sleeves, one on top of the other to prevent any leakage.

A typical expansion joint of this type is presented in figure 10.4

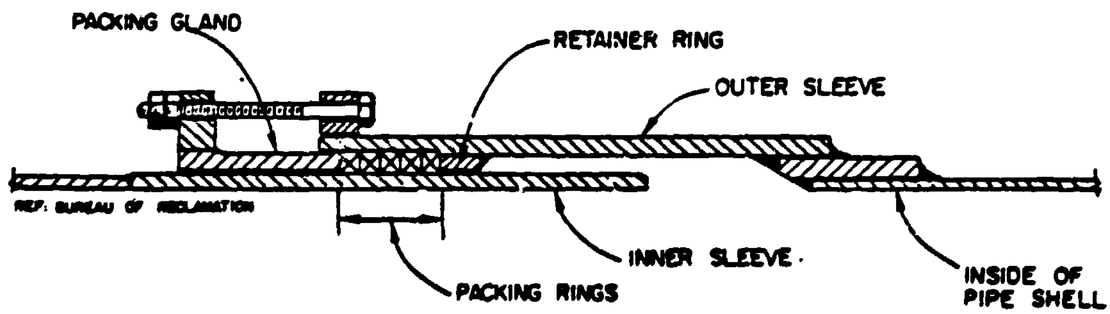


Fig. Nr 10.4 Sleeve type expansion joint

This type of joint can be designed with two stuffing sides to allow transverse deflexions and temperature dilatation movements, see figure 10.5.

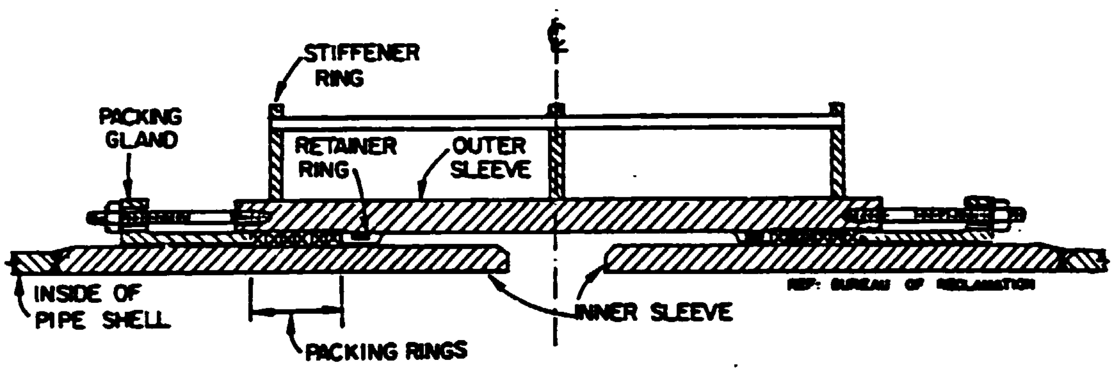
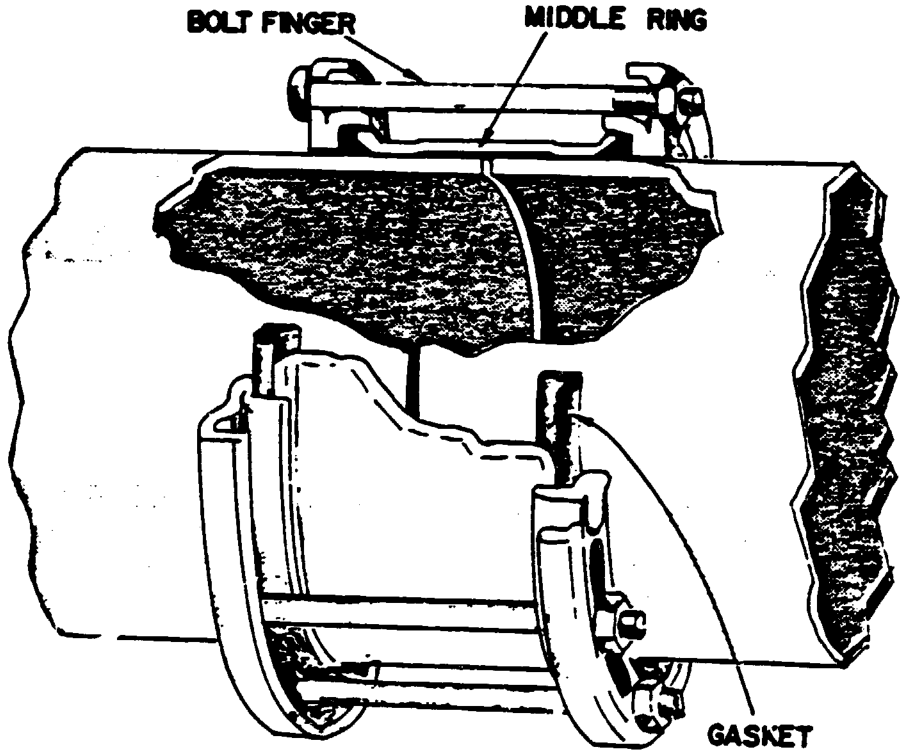


Fig. Nr. 10.5 Double sleeve type expansion joint

Another way used for of constructing the penstock includes the use of Dresser Coupled expansions, which are a patented system that allows an easy construction with a factory-made stress-free joint. With this type of sleeve locked up stresses caused otherwise by field-welded joints, are inexistent. Construction time can be sped up considerably.

In figure 10.6 a schematic view of a dresser coupling is presented.



REF.: DRESSER CATALOG

Fig. Nr. 10.6 Schematic working principle of a dresser coupling

10.8 Anchors

All welded steel penstocks lines freely supported above the ground surface or in tunnels must be provided with anchors at bends and at intermediate points in long straight lenghts.

The purpose of the anchors is to fix the penstocks in place during installation and during operation. They resist the various forces acting on them. During installation, with the penstock empty, only temperature and gravity forces need to be considered. The forces active on a pipe bend when the line is in operation consist mainly of:

- ◇ Temperature forces
- ◇ Hydrostatic forces
- ◇ Dynamic forces
- ◇ Gravity forces

Anchors are generally not required for buried pipe except at horizontal bends with large deflection angles and at vertical or overbends with high uplift forces which can not be resisted by the backfill alone.

The combination of forces on an anchor tend to overturn it or to slide along its soil foundation. If the anchor is placed completely around a pipe in a continually welded line, overturning really cannot take place. Such anchors should therefore be installed primarily to safely resist the sliding forces.

In countries of seismic activity, earthquake forces should be included in the designs as recommended by the local engineering codes.

The distance between anchors should be of the order of 50 - 80 m . Anchor blocks and saddles are to be constructed in reinforced concrete.

The penstock in general should be protected against rock and tree falls. This protection can usually be achieved with the construction of protection trenches, walls, wire mesh, etc.

It is recommended to install stairs on the steep parts to facilitate the construction and the future maintenance

For the Civil works execution and later for the erection of the steel works a winch is convenient to be mounted parallel to the penstock.

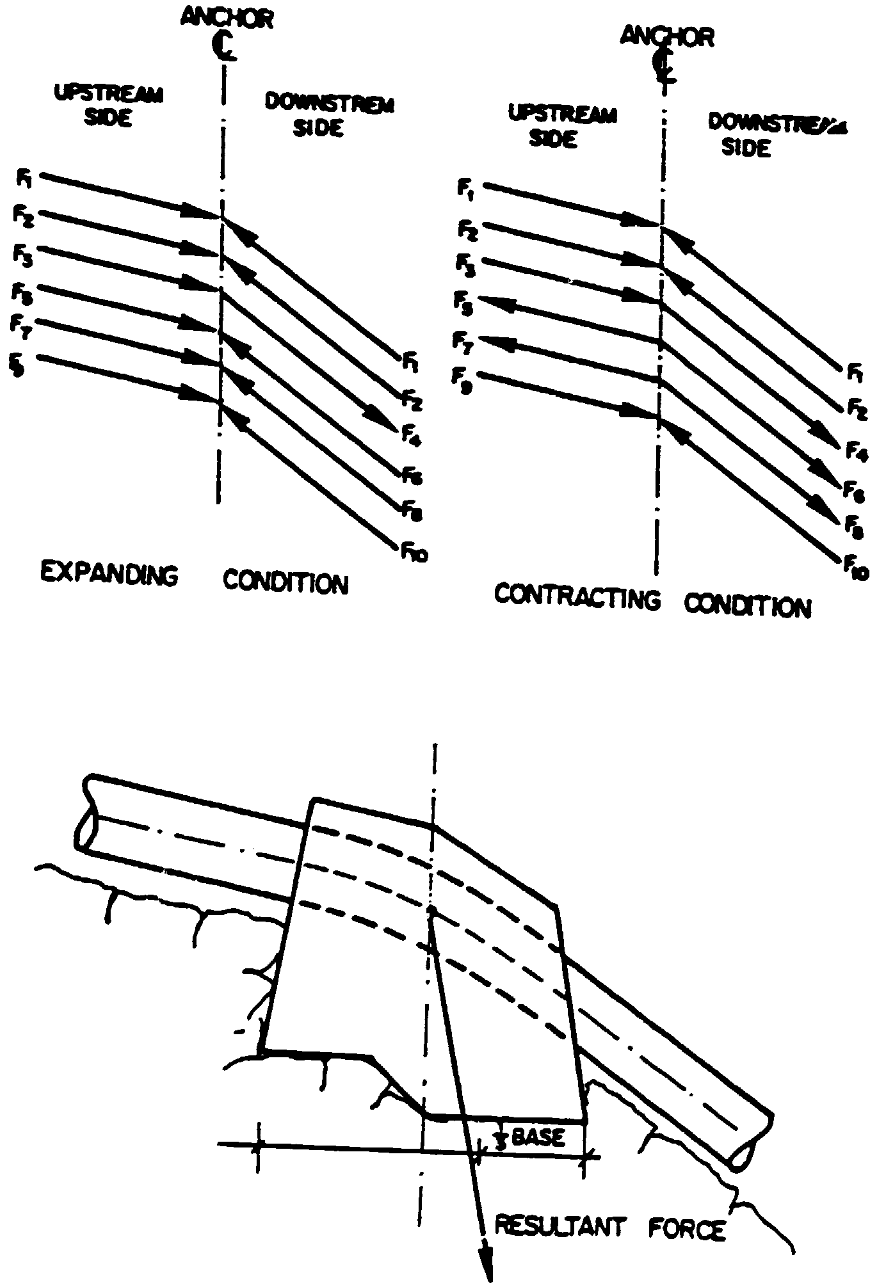


Fig. Nr. 10.7 Forces acting in an anchor block

The main forces acting in an anchor block are the following (see Fig. 10.7):

1. **The hydrostatic force** that acts along the pipe axis at both sides of the bend:

$$F_1 = \gamma A H$$

where: γ = Weight of water, = 1000 kg/m^3
A = Cross section area (m^2)
H = Maximum water head (m)

2. **The dynamic forces** acting against the outside bend:

$$F_2 = \frac{\gamma Q v}{g}$$

γ = weigth of water = 1000 kg/ m^3
Q = Flow of water (m^3/s)
v = velocity of water (m/s)
g = acceleration of gravity = 9.81 m/s^2

3. **Force due to dead weigth** of the pipe, from anchor uphill to expansion joint, tending to slide downhill over piers.

$$F_3 = P_1 \sin x$$

P_1 = Dead weight of pipe from anchor uphill to expansion joint in kg.
x = slope angle above anchor.

4. **Force due to dead weight** of pipe from anchor downhill to expansion joint, tending to slide downhill over piers.

$$F_4 = P_2 \sin y$$

where: P_2 = Dead weight of pipe downhill from anchor to expansion joint in kg.
y = slope angle below anchor.

5. **Sliding friction of pipe** on piers due to expansion or contraction uphill from anchor:

$$F_5 = f_5 \cos x \,(P_1 + W_1 - p_1/2)$$

the coefficient f_5 of friction of pipe on piers can be taken as:

Steel on concrete $f_5 = 0.60$
Steel on steel(rusty) $f_5 = 0.50$
Steel on steel (greasy) $f_5 = 0.25$

x = slope angle above anchor.
P_1 = Dead weight of pipe from anchor uphill to expansion joint (kg)
W = Weight of water in pipe P_1 (kg)
p_1 = weight of pipe and contained water from anchor to adjacent uphill pier in kg

6. **Sliding friction of pipe** on piers due to expansion or contraction downhill from anchor:

$$F_6 = f_6 \cos y \,(P_2 + W_2 - p_2/2)$$

f_6 = coefficient of friction of pipe on piers = f_5
P_2 = Dead weight of pipe downhill from anchor to expansion joint in kg.
W_2 = Weight of water in pipe P_2
p_2 = weight of pipe and contained water from anchor to adjacent downhill pier in kg.

7. **Sliding friction** of uphill expansion joint:

$$F_7 = f_7 \,\pi\, (D + 2\, t)$$

where:

f_7 = friction of expansion joint per linear meter of circunference approx. equal to 250 kg / ml
D = inside diameter of pipe (m)
t = wall thickness of pipe shell (m)

8. **Sliding friction** of downhill expansion joint:

$$F_8 = f_8 \ \pi \ (D + 2t)$$

where: $f_8 = f_7$

9. **Hydrostatic pressure** on exposed end of pipe in uphill expansion joint:

$$F_9 = \gamma H \ \pi \ t_9 \ (D + t_9)$$

where:
γ = weight of water = 1000 kg/m3
D = Pipe diameter (m)
H = Maximum water head (m)
t_9 = wall thickness of pipe wall (m)

10. **Hydrostatic pressure** on exposed end of pipe in downhill expansion joint:

$$F_{10} = \gamma H \pi t_{10} (D + 2 t_{10})$$

where:
γ = weight of water = 1000 kg/m3
D = Pipe diameter (m)
H = Maximum water head (m)
t_{10} = wall thickness of pipe wall (m)

All the above-mentioned forces should be considered in an expanding and then in a contracting condition and the resultant of all these forces will be combined with the weight of the anchor block. The resultant force has to pass through the 1/3 of the base of the anchor.

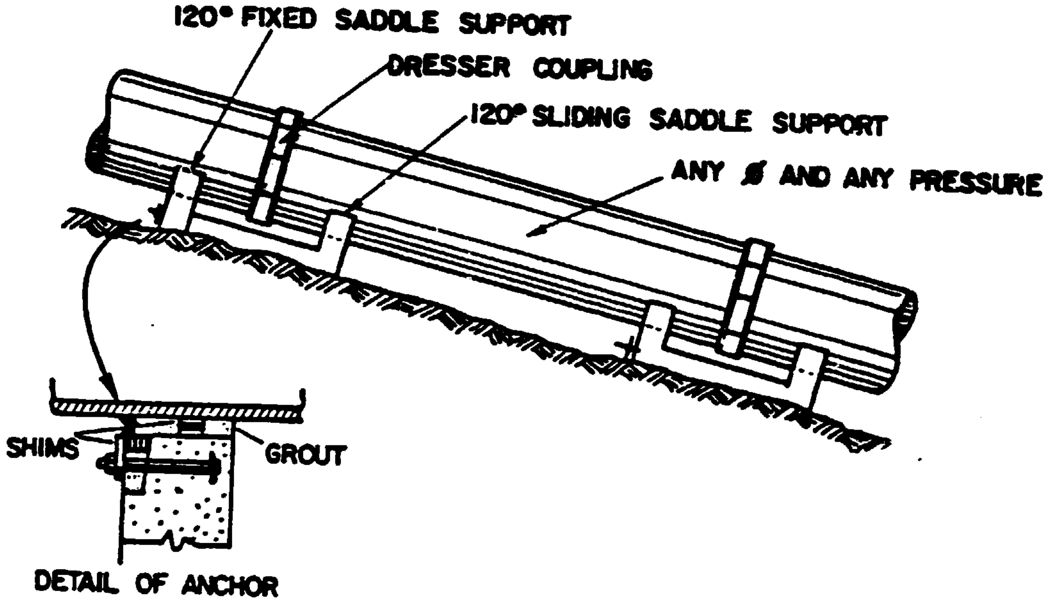
In countries were seismic activity requires consideration of earthquake, the corresponding forces must be added. Forces due to expansions or contractions of the pipe have not been considered in the above-mentioned system.

In figure 10.8 a schematic diagram of the forces acting on an anchor block are depicted.

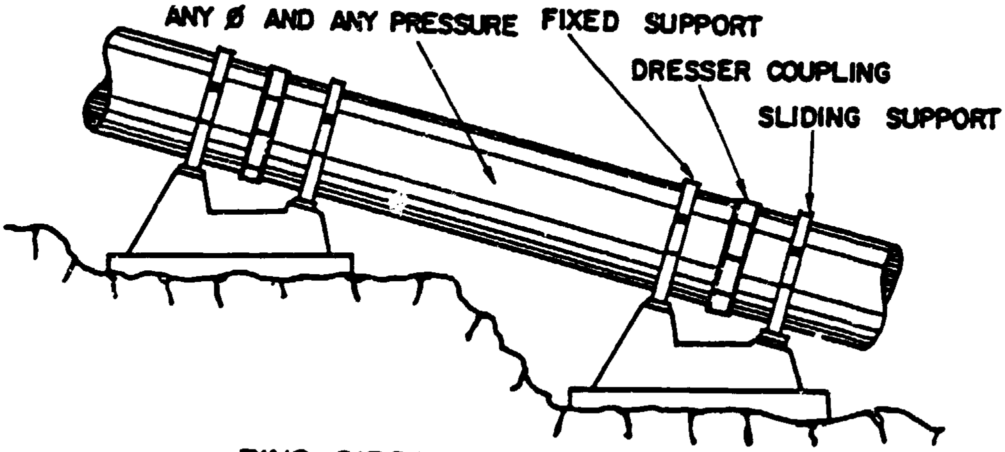
The anchor block should be calculated to resist sliding and for the allowable bearing capacity of the soil.

In figures 10. 8 and 1C.9 schematic support of pipes joined with Dresser couplings are depicted. A Ring girder type support and a 120 ° saddle type support will be seen in fig 10.9 which apply for steep hill sides or in very rough terrain.

In fig 10.9 intermediate supports for small diameter pipes is presented, center type supports, and "two and one" type support to be use with the combination of Dresser coupling.



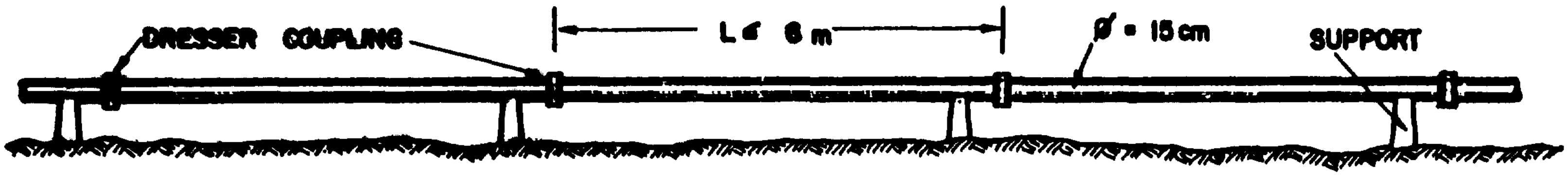
120° SADDLE TYPE SUPPORT



RING GIRDER TYPE SUPPORT

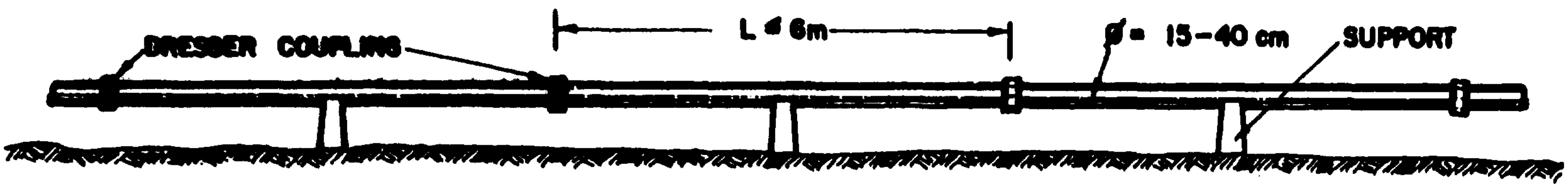
(SUPPC'RTS FOR STEEP HILL SIDES OR VERY ROUGH TERRAIN)
REF.: DRESSER CATALOG

Fig. 10.8 Support of pipes joined with dresser coupling



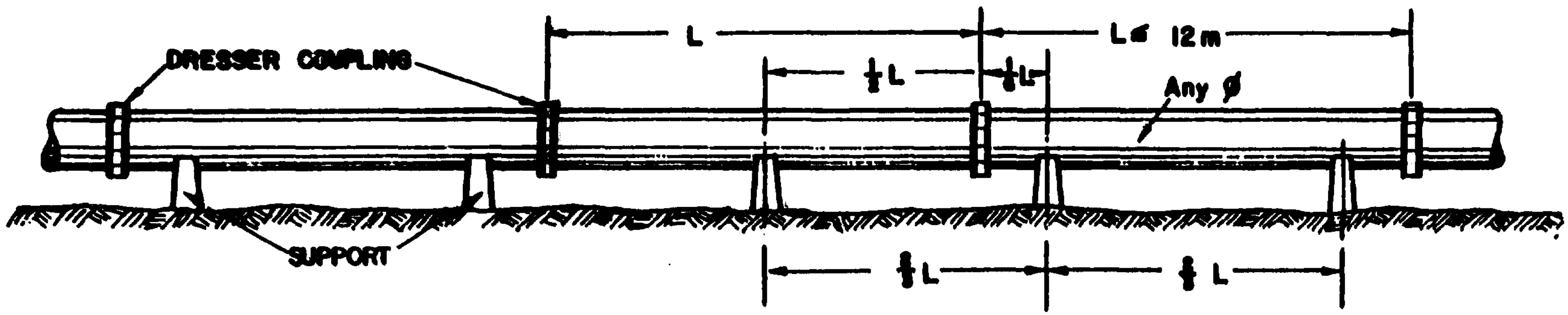
SUPPORT FOR SMALL DIAMETER ONLY

SUITABLE FOR ANY PRESSURE PROVIDED PIPE IS WELL ANCHORED TO SUPPORT THE PRESSURES



CENTER TYPE SUPPORT

SUITABLE FOR PRESSURES UP TO 12 kg MAXIMUM. PIPE MUST BE WELL ANCHORED TO SUPPORTS



REF. : DRESSER CATALOG

"TWO AND ONE" TYPE SUPPORT

SUITABLE FOR ANY PRESSURE PROVIDED ADEQUATE ANCHORING IS FORESEEN.
TO BE USED ONLY ON TERRAIN WHICH IS LEVEL OR OF SMALL SLOPE.
EACH LENGTH OF PIPE MUST BE ANCHORED TO ONE (AND ONLY ONE) SUPPORT

Fig. 10.9 Some types of intermediate supports

Chapter 11

11. Powerhouse

As its principal purpose, the powerhouse building is to protect the generating equipment and as such it has to be built in as protected place as possible, i. e. it must be protected from landslides, from river floods and other natural hazards.

The foundation of the powerhouse should be designed to avoid soil settlements; for that reason, it is recommended to provide a good drainage system to the foundation soil able to maintain at all times dry conditions of the ground below the foundation slab.

It is convenient to provide a permanent access to the powerhouse.

The size of the powerhouse should be sufficient to accommodate the required equipment, with sufficient free space along the installation to permit easy operation and maintenance.

The main door of access should have a width of at least 0.60 m wider than the largest equipment part, to allow for its repair and transport if necessary.

For the manipulation of the heavy turbogenerating machinery it is convenient to install a permanent hoisting equipment. The lowest point of the hoist should be such to allow to lift the largest piece of equipment and still have a free margin of 0.50 to 0.60 m

In the example used in this report, it has been assumed that reinforced concrete is available, however depending on the country and its location, other materials locally available could be used. In any case, a solidly constructed powerhouse warrants a longer life and is in the longer term less expensive, as it will require less maintenance.

The roof should be adapted and designed in accordance with local conditions, depending whether protection against precipitation of water and snow or water alone has to be provided. The roof may also serve only as protection against the solar radiation; final decisions have to be made according to the local conditions and in situ.

Below the powerhouse, provisions should be made for the discharge of the turbinated water, in a channel that comunicates directly with the tailrace channel. There are small differences of the transition zone from the turbines to the tailrace channel, depending on the type of turbine to be used, Pelton, Francis or Cross-flow turbines. However the differences are small and can be adapted easily to different alternatives.

The tailrace self is a channel that can be treated in most cases as the channels mentioned in chapter 8 of this report.

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WORKED OUT

EXAMPLES

EXAMPLE 1

Conventional Water Intake

(Without Desilting Basin)

$Q = 1\ m^3/s$

EXAMPLE OF A CONVENTIONAL WATER INTAKE FOR Q= 1 m^3/s

An example of a conventional water intake for a Q of 1 m^3/s is presented in Drawing Nr. 40093-1-1, 1-2 and 1-3

This example present a water intake with its foundation in a favourable river bed (solid rock). For the purposes of this example, a normal transport of sediments has been assumed and that this transport generally occurs mainly during floods.

A maximum design flood (Q_{Max}) of 12 m^3/sec has been adopted which corresponds to an approximately 1000 years flood.

The intake and the sluice channel have been located on the external bend of the river which provides a better catchment of clear water without sediments, and an easy removal of the sediments through the sluice channel.

Aprons on the inside part of the river should contribute to improve the flow pattern into the intake; a fixed weir on the inside part of the river bend completes the whole intake system.

The weir, the intake structure and the spillway as well as the sluice channel have been planned to be built in masonry, which for this type and size of construction is universally well accepted, is cheaper to build and has demonstrated a good performance in several similar plants built elsewhere around the world.

The part corresponding to the gate and to the trash rack have been planned to be build with light reinforced concrete PC-300.

The sluice have been designed with the following main parts:

- An inlet sill (that favours the flow of sediments)
- A sluice channel with a gradient of S = 0.08 in order to ensure the proper hydraulic regimen and that sediments will not clog-up this important element.
- A sluice gate of 1.00 m width and 1.00 m height with mechanical operation, which can be hand operated but may be better equiped with a small electric motor.
- With the gate open, the capacity of the sluice channel allows to pass a maximum free flow flush of 3 m^3/sec, considered as appropriate and sufficient for this type of intake; however this should be checked in each case in accordance with the local hydrologic conditions, and adapted if the flow is larger.

The trash rack should have an inclination of about 15° with the vertical, in order to allow maintenance, to easily clean the rack.

The dimension of the trash rack is of 2.50 m width and 0.80 m height, and the arrangement of the bars should be as depicted in Fig. Nr. EX1-1

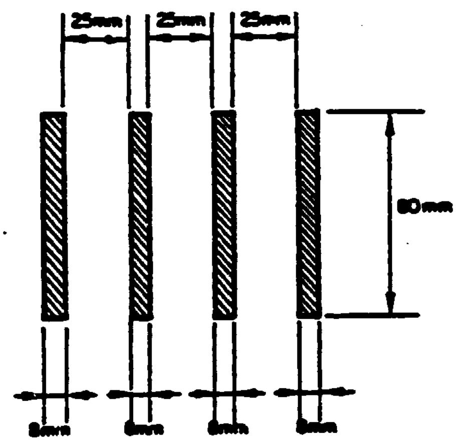

FIG. Nr. EX 1-1 Detail of trash rack steel bars

The gross velocity of the flowing water before the bars have been assumed as 0.50 m/sec which is a conservative value confirmed by the experience in other similar intakes (This velocity ensures also that a minimum of sediments will pass through that rack).

The arrangement of the sluice channel and the trash rack permits to have an appreciable amount of sediments deposit in front of the intake and to ensure safe intake operation with the necessary margin of security.

The depth and the amount of sediments have to be ckecked regularly in order to provide periodical flushing of sediments through the sluice channel. It is important to specify those flushing operations in the operation manual for the maintenance personnel.

The gate should be operated at regular intervals, inclusive when no sediments are present, with the aim to assure its proper operation at any time. A gate should not be allowed to remain unoperative for a longer period of time thus avoiding its blocking.

In case of necessity of inspection or reparation of the flush gate, a slot for placing stop logs made of horizontal elements has been provided. The stop logs can be installed during a period of low water flow in the river.

The intake structure is provided with aeration and access opening.

At the entrance of the intake, slots for stop logs have been provided as protection of the adduction canal during canal inspections. It is pertinent to mention that during those inspections the sluice gate should be maintained in the open position.

A control section of 0.60 m x 0.60 m permits to reduce any excessive water inflows during highflows, see Drawing Nr.40093-1-3 numeral 13.

A small sluice gate of 0.20 m x 0.30 m allows to clean any sediments that inspite of the security devices provided could pass through and into the canal, see Drawing Nr. 40093-1-2 and 1-3, numeral 14.

The Adduction canal has an extra built in safety which consist of a lateral spillway, with a width of 4.00 m. It will take care of any excess of water that could occur during floods, avoiding any danger of overflow of the canal.

The crest elevation is on level 199.70 m a.s.l. A sill and flood protection beam see Drawing Nr.40093-1-3 numeral 15, will cut any excess of water, forcing the water excess to spill through the lateral spillway.

The bottom of the canal has been set at the elevation 199.00 m a.s.l., the slope of canal has been adopted as S = 0.0012, in order to provide a good subcritical flow regime (it is not recommended to use a gradient on the sluice channel bottom, of less than S = 0.001).

The bottom width of the canal for a normal design discharge of Q = 1 m^3/s is 1.35 m made of concrete type PC-300 and inclined walls made of masonry of a 5:1 inclination of the horizontal to the vertical (The maximum water depth should be 0.70 m).

The velocity of the water has been assumed to be of 1.10 m/s.

CONVENTIONAL WATER INTAKE FOR Q = 1 m^3/s

LEGENDE

1. SLUICE SILL EL. 198.00 m a.s.l. IN MASONRY
2. SLUICE CHANNEL S = 0.08
3. STOP LOGS SLOTS
4. GUIDE WALL CREST EL. 200.60
5. SLUICE GATE 1.00 m x 1.00 m
6. NORMAL RETENTION EL 200.00 m a.s.l.
7. WEIR IN MASONRY CREST EL 200.10 m a.s.l.
8. APRON IN MASONRY
9. INTAKE WING WALL IN MASONRY
10. TRASH RACK 2.50 m X 0.95 m
11. AIREATION AND ACCESS OPENING
12. FLOOD PROTECTION WALL AND OPERATION PLATFORM.
13. CONTROL SECTION 0.60 m X 0.60 m
14. SLUICE GATE 0.20 m X 0.30 m
15. SPILWAY W = 4.00 m
16. FLOOD PROTECTION
17. ADDUCTION CANAL Q = 1 m^3/s
18. 1000 YEAR FLOOD WATER LEVEL

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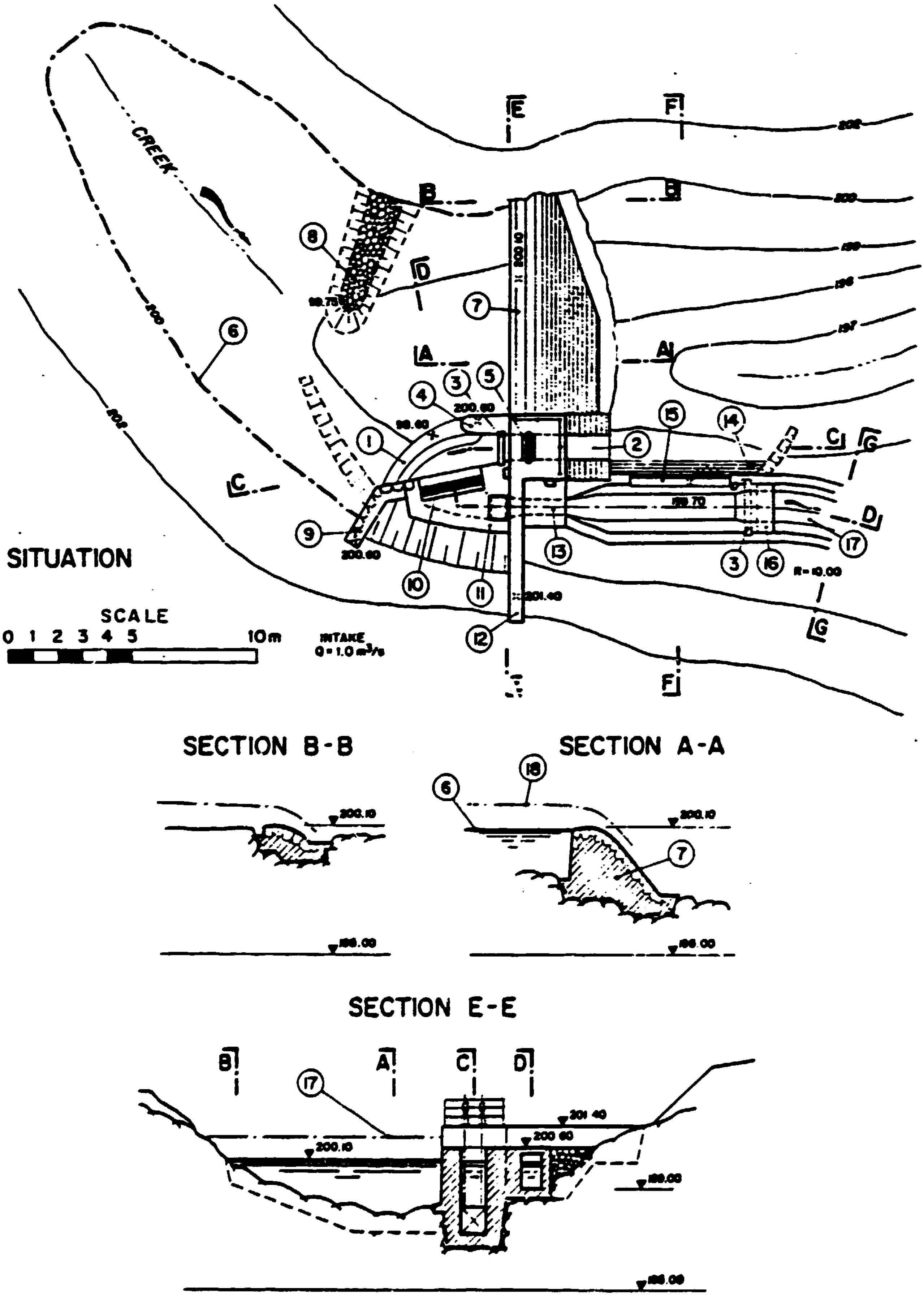

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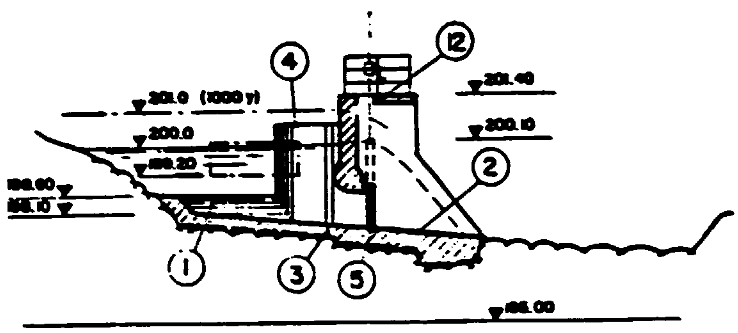
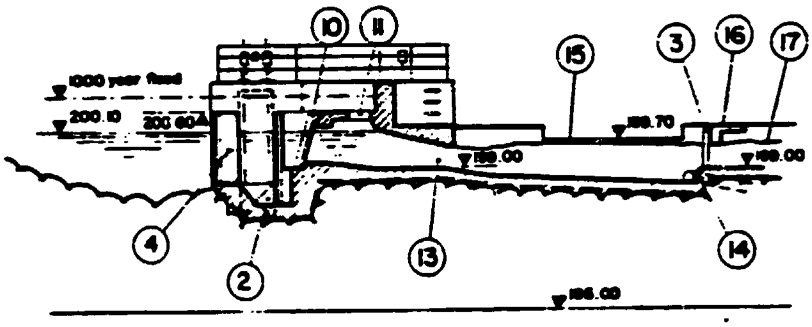
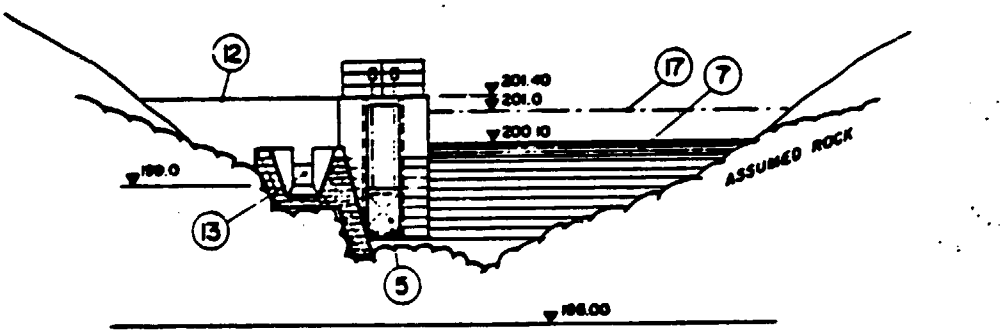
SECTION C-C



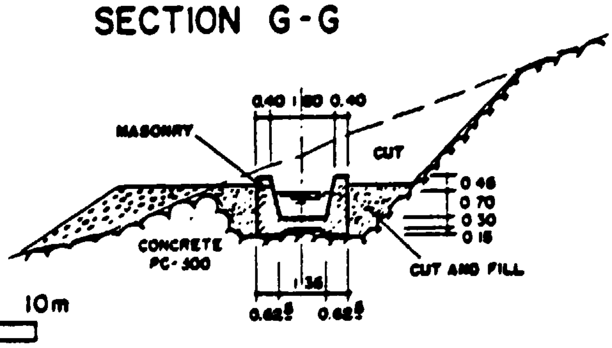
SECTION D-D



SECTION F-F



SECTION G-G



SCALE
0 1 2 3 4 5 10m

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EXAMPLE 2

Conventional Water Intake

with Desilting Basin

Q = 2.6 m3/s

EXAMPLE OF A CONVENTIONAL WATER INTAKE OF $Q = 2.6\ m^3/s$

An example of a conventional (run-of-river) intake for a $Q = 2.6\ m^3/s$ is presented in Drawing Nr.40093-2-1 / 2-5

This example present a water intake with its foundation located in a favourable river bed at the end of a river section with small gradient and gentle slopes. The foundation is located on gravel and coarse sand. It represent an appropriate solution for a wide valley width, in which part of it has to be closed with a long, side dam.

It has been assumed that there are no large transports of sediments on the river bed, but only some suspended fine sediments are carried by the river.

Intake weir and dam These three structures have been designed in generally with a reinforced concrete of the type PC - 300.

The weir has been designed to allow the passage of a 1000 year flood, for which it has been assumed that the water discharge is of the order of 22 m^3/s, adopted as a normal average when compared with similar projects of this type and size.

After the weir, a small stilling basin has been designed. The bottom protection is given with geotextiles and gabions.

On the right side there is a retaining wall with a face of 4 : 1 (vertical to horizontal) which is a favourable slope for an earthfill dam.

It is necessary to obtain samples from pits and trenches in this type of soils, in order to be able to define the soil properties and make the appropriate dam design, as well as to determine the most suitable design criteria.

The design criteria of small dams is not covered in this report, it is beyond its scope, However, the authors refer to the readers interested in this aspect of a project to consult the book published by the Bureau of Reclamation of The United States of America, entitled "Design of Small Dams".

The sluice channel have been conceived to pass a discharge of 1 - 4 m^3/s with free flow and supercritical conditions, for which it is necessary to give to the bottom of the sluice channel a slope of $S = 0.08$ to provide the above-mentioned conditions and to guarantee keeping the sluice channel always free of sediment deposits and ready to operate at any time.

The design of the sluice channel ensures the keeping clean of the area in front of the intake entrance. Each time the sluice gate is open, the sediments in front of the trash rack will be flushed away through the sluice channel provided enough water discharge is available.

The sluice gate (see Drawing Nr.40093-2-2 and 2-4, numeral 10), has to be put into periodical operation regularly in order to guarantee a proper sluice operation at any time, and avoid undesirable surprises in an emergency case. In front of the sluice gate it is forseen and recommended to install a stop log groove that will allow closing that part if repairs are to be made on the gate.

At the intake, the sill has been designed high enough to provide for a excellent protection against the entrance of gravel in the intake canal.

The trash rack is to be placed with an inclination of 75°. It should be constructed with steel bars, of 8 mm width and 80 mm height, spaced with opennings of 25 mm between bars.

The gross water velocity at the trash rack should not exceed 0.60 m/s and the net velocity not exceed 0.80 m/s.

The intake gate (see Drawing Nr.40093-2-1 and 2-4, numeral 16) is dimensioned 1.00 m in width by 1.00 m in height (the small section is necessary as a restriction in case of floods).

After the entrance, tranquilization racks should provide a reduction of the water velocity into the desilting basin, constructed immediately after the entrance canal.

The desilting basin for a conventional water intake for a Q = 2.60 m^3/s, is designed to be able to retain all granular material which has managed to pass the trash rack, with diameters of 0.20 - 0.40 mm. The apron water velocity is of the order of 0.25 m/s, which means that this basin can clean all particles with diameter bigger than 0.20 mm.

Particles with diameter bigger than 0.20 mm are known to be dangerous for the equipment, and especially for the turbines if they are of the Francis or Pelton type; turbines of the Crossflow type are less sensitive to this type of particles.

The main dimensions for a discharge design of Q = 2.6 m^3/s are the following (see Table 7.1 too):

$$Q = 2.60\ m^3/s$$
$$B = 3.50\ m$$

L = 28.00 m
D = 3.10 m
W = 1.10 m

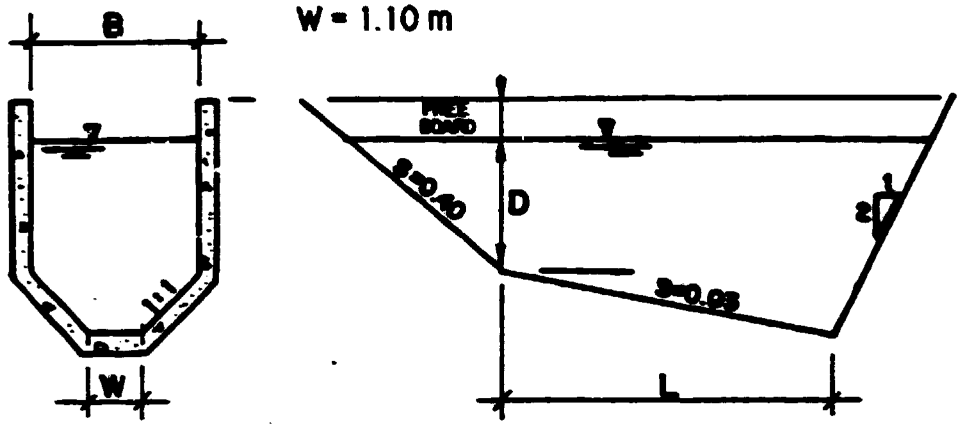

Fig. Nr. Ex 2-1 <u>Main dimensions of the Desilting Plant for Q = 2.60 m3/s</u>

The Adduction canal has been designed with the following main dimensions (taken from Table Nr. 8.3):

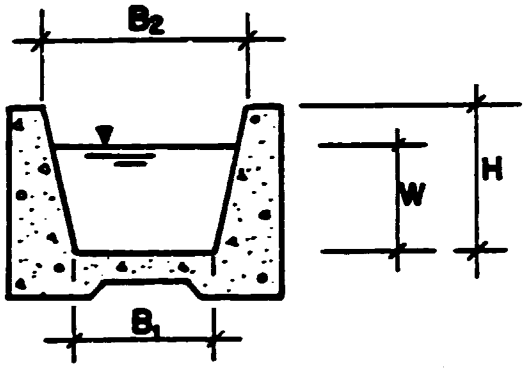

FIG. Nr.Ex.2-2 **Cross section of canal example for Q = 2.60 m^3/s**

Q = 2.60 m^3/s
B_1 = 1.85 m
B_2 = 2.45 m
W = 1.05 m
H = 1.55 m

The radii in the curves are recommended not to be chosen smaller than 10 - 12 m , to avoid unnecessary spilling of water , or to avoid complicated curved sections with varied lateral gradients.

The slope of the canal should be on the order of 0.0012, with a concrete bottom (PC - 300), with light reinforcement of steel bars; the walls of the canal can be made of masonry.

CONVENTIONAL WATER INTAKE for $Q = 2.6\ m^3/s$

LEGENDE

1. WEIR CREST EL. 200.10 m a.s.l.
2. COMPACTED CORE MATERIAL
3. SELECTED FILL COMPACTED
4. GEOTEXTILE
5. GABIONS 2.00 m x 1.00 m x 0.75 m
6. LEFT RETAINING WALL
7. SLUICE SILL EL. 198.45
8. SLUICE CHANNEL $S = 0.08$
9. GROOVE FOR STOP LOGS
10. SLUICE GATE 1.60 m x 1.60 m
11. RETENTION: NORMAL LEVEL 2000.00 m a.sl. WITH THE SLUICE GATES OPEN DURING A 1000 YEAR FLOOD
12. GUIDING WALL EL. 201.00 m a.s.l. WITH THE SLUICE GATES CLOSED.
13. TRASH RACK 4.60 m x 1.10 m OPENING SPACE BETWEEN
    BARS 25 mm, INCLINATION OF RACK 75 °
14. AERATION AND ACCESS OPENING
15. FLOOD PROTECTION WALL AND OPERATION PLATFORM
16. ENTRANCE GATE 1.00 m x 1.00 m
17. HANDRAILS
18. STAIR
19. SLUICE 1.10 m x 1.30 m
20. FLUSH GATE 1.00 m x 1.00 m
21. STOP LOGS GROOVE
22. SPILLWAY
23. FLOOD SPILLWAY, CREST EL. 198.95 m a.s.l.
24. RIP - RAP PROTECTION Ø = 0.90 m
25. FLOOD PROTECTION
26. ADDUCTION CANAL $Q = 2.60\ m^3/s$

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27. EXPANSION JOINT WITH WATERSTOP
28. SUPPORT SLAB
29. TRANQUILIZATION RACKS.

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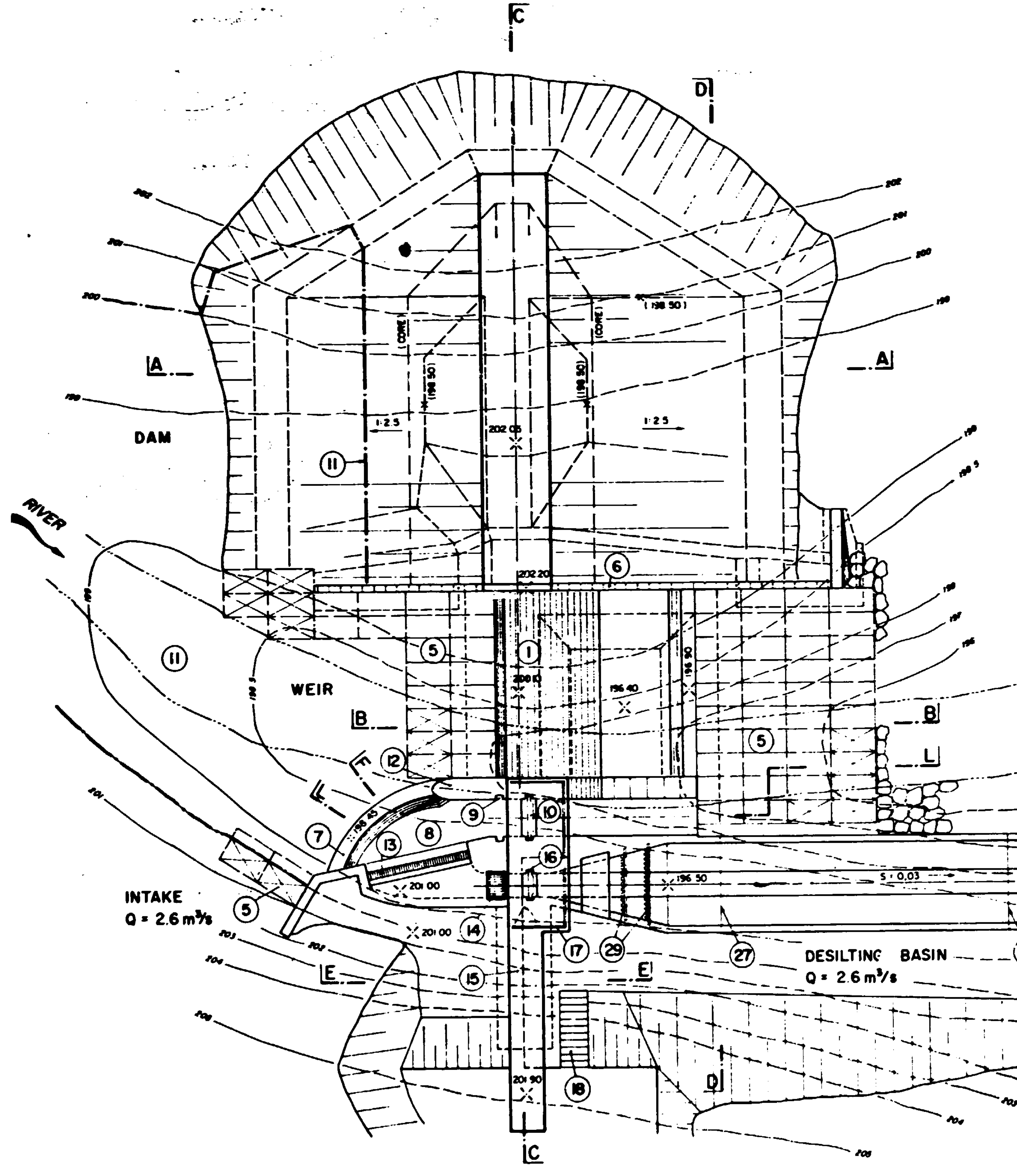
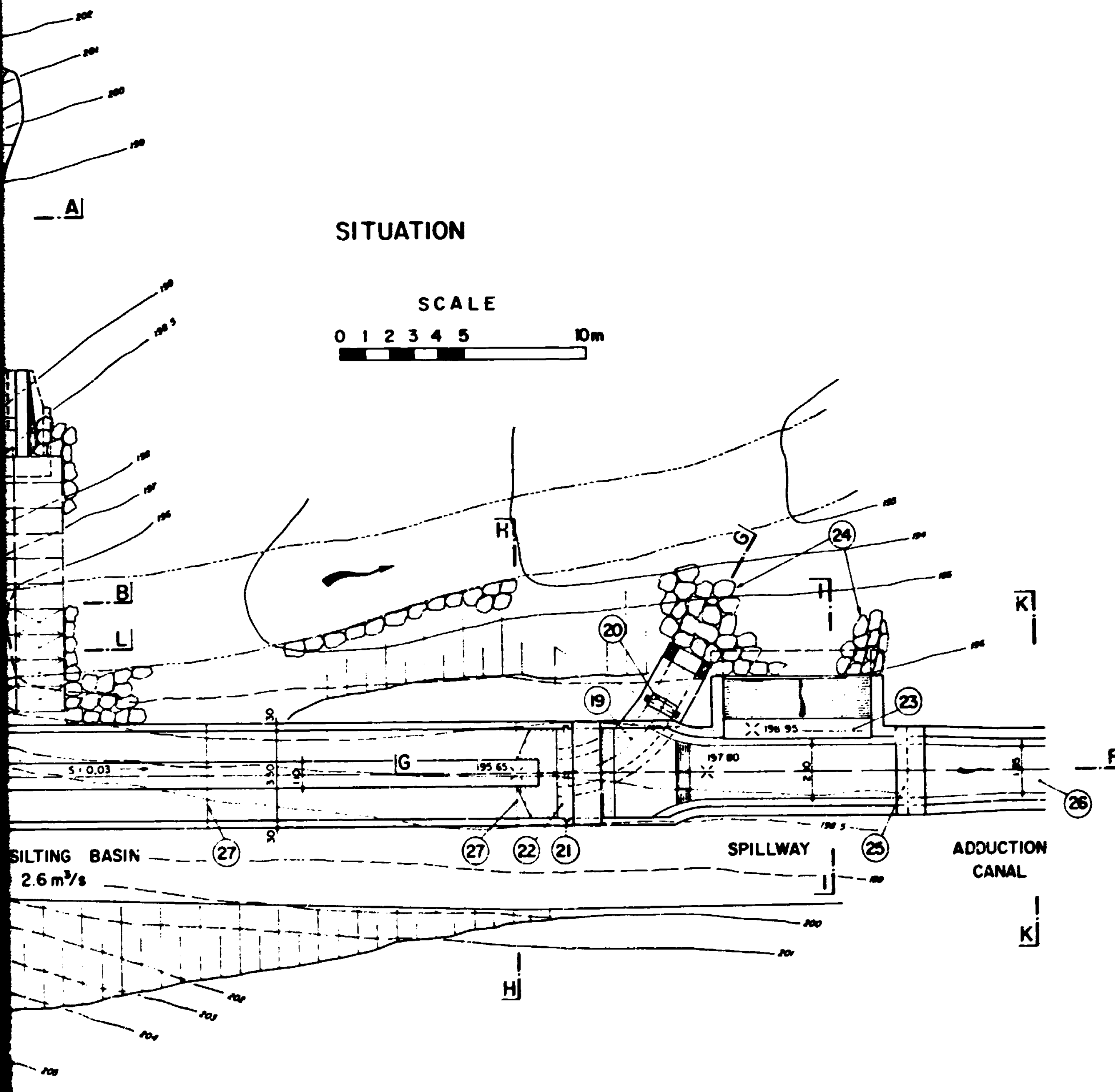


DAM
WEIR
RIVER
INTAKE
Q = 2.6 m³/s
DESILTING BASIN
Q = 2.6 m³/s
(CORE)
1:2.5
202.05
202.20
200.10
196.40
196.50
201.00
201.90
198.45
S = 0.03

SECTION 1

SITUATION

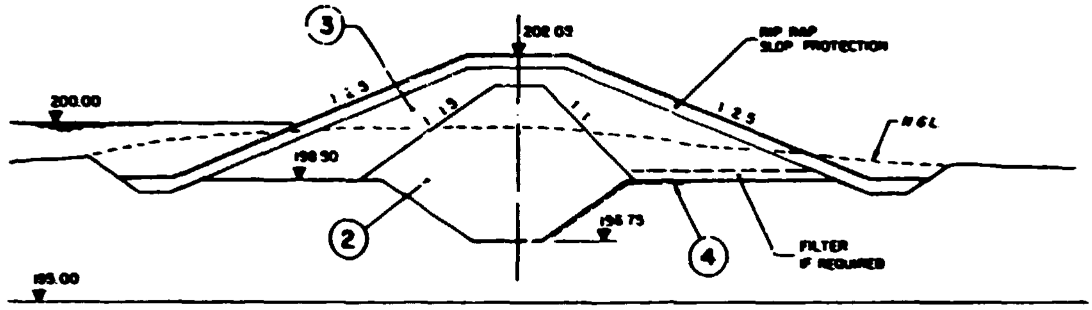
SCALE



SECTION 2

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 2
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. ZUA DESIGN BU APP KRU	DRAWING NR 40093-2-2 APRIL 1987

SECTION A - A



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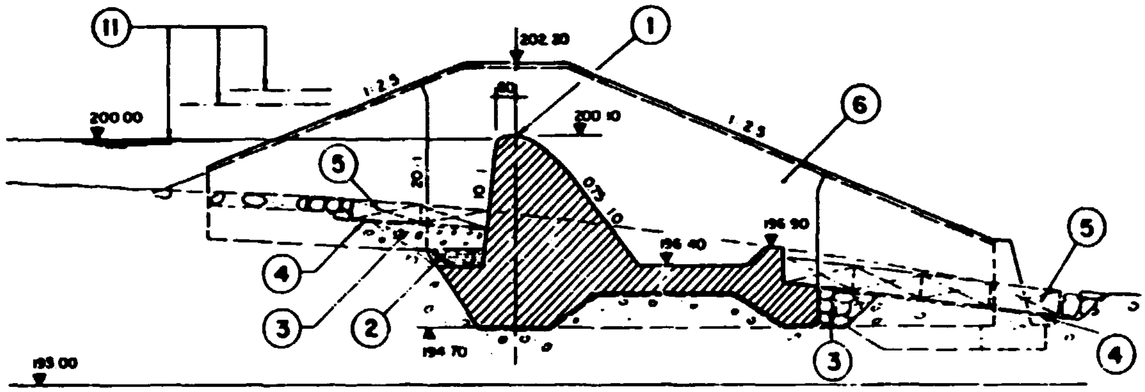

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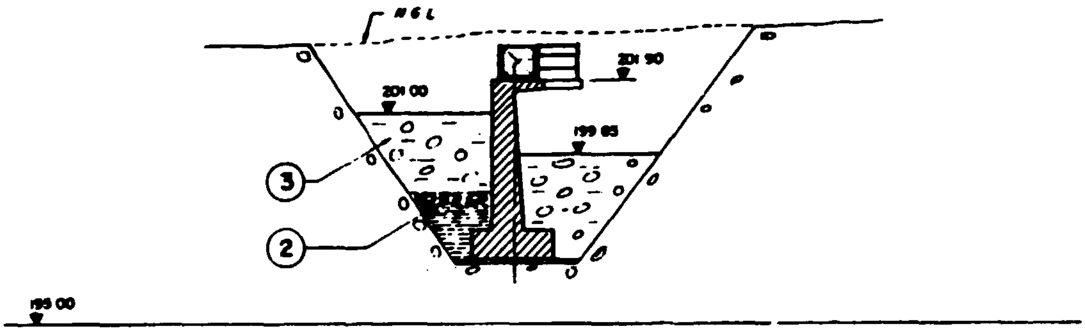

SCALE
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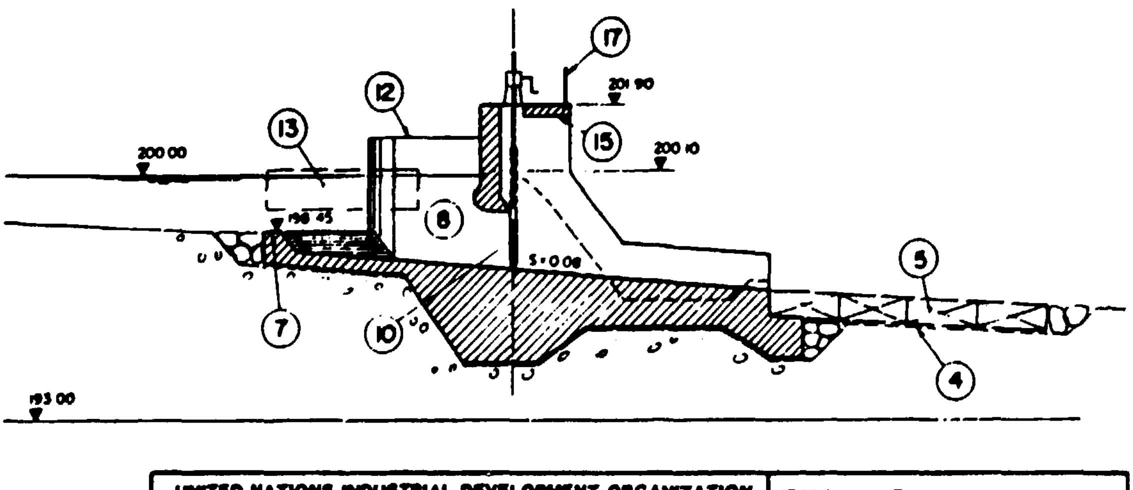
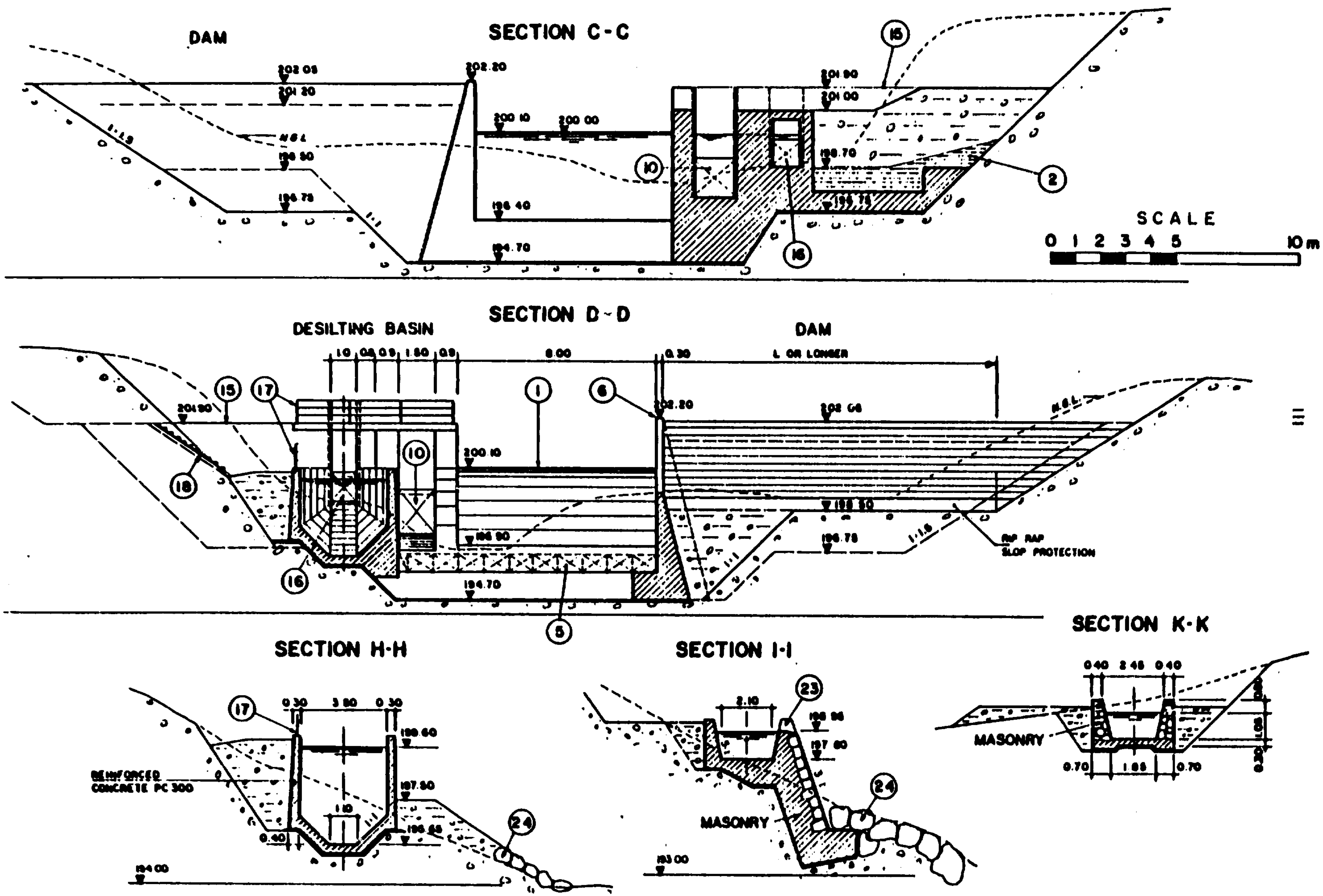
SECTION L - L



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 2
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. ZUA DESIGN BU APP. KRU	DRAWING NR 40093-2-3 APRIL 1987



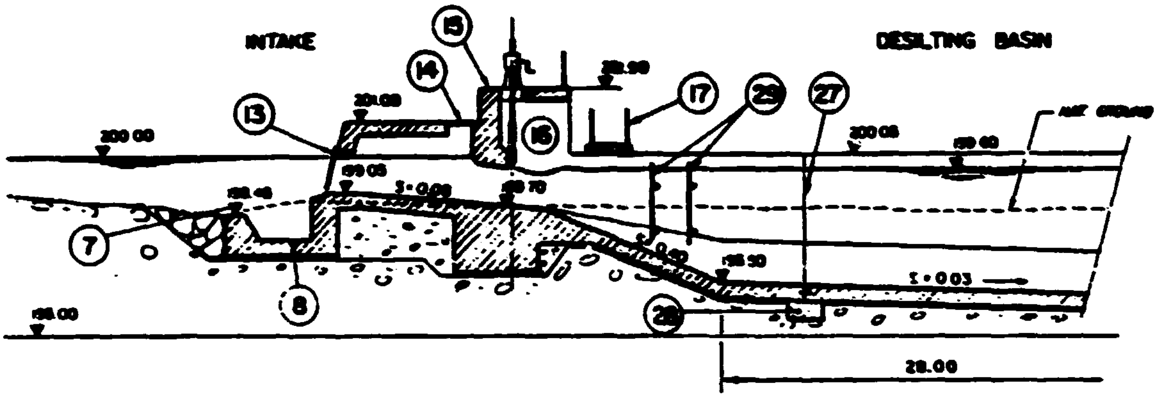
UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION

ELECTROWATT
ENGINEERING SERVICES LTD.
ZURICH

DRA.	HOL.
DESIGN	SU
APP.	KRU

EXAMPLE 2
DRAWING NR 40083-2-4
APRIL 1987

SECTION F - F



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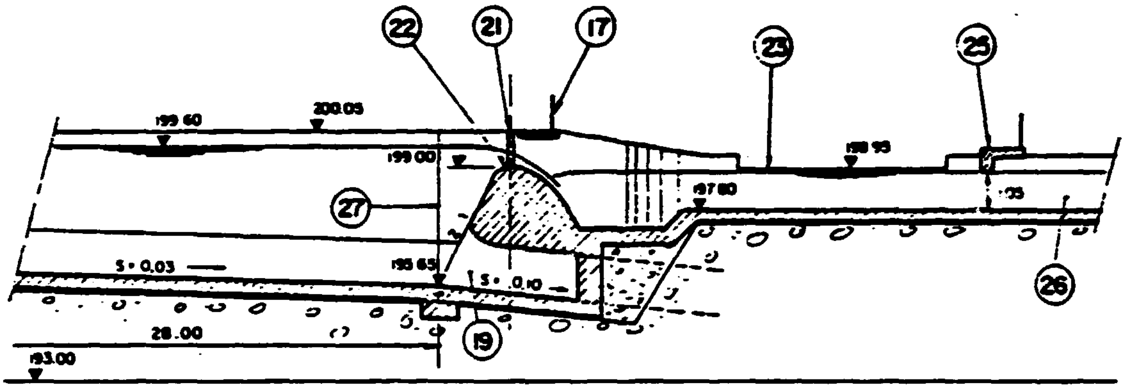
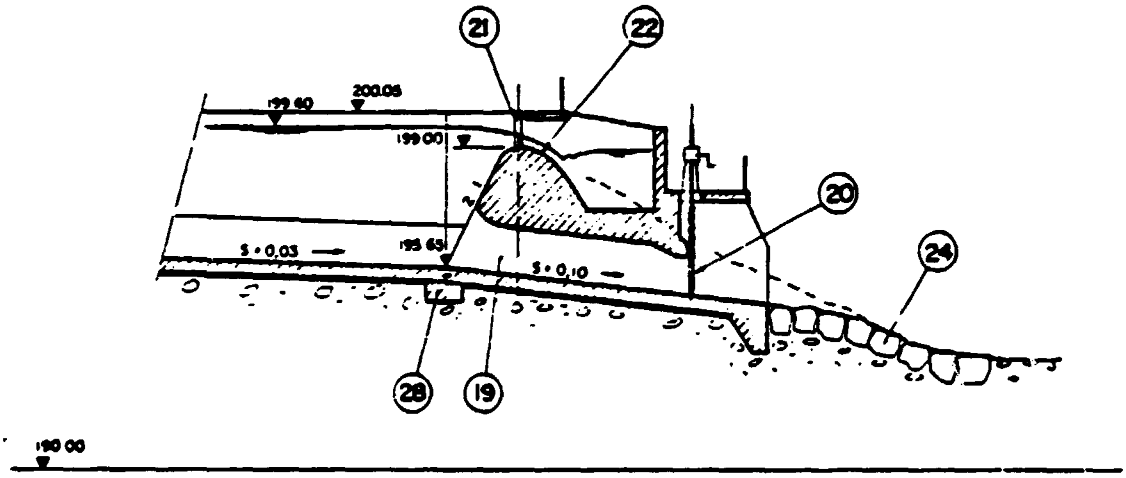


SECTION G - G



SCALE
0 1 2 3 4 5 10m

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 2
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA.: ZUA DESIGN: BU APP.: NRU	DRAWING NR 40093-2-5 APRIL 1987

EXAMPLE 3

Conventional Water Intake

with Desilting Basin

$Q = 3.5\ m^3/s$ 3

EXAMPLE OF A CONVENTIONAL WATER INTAKE FOR $Q = 3.5\ m^3/s$

An example of a conventional Water Intake (run-of-river type) for $Q = 3.5\ m^3/s$ is presented in Drawing No. 40093-3-1 / 3-4.
The foundation of the structures has been assumed in a favourable river bed of sound rock.

For the design floods, the following values have been adopted:

Q (10 years) = 18 m^3/s

Q (50 years) = 24 m^3/s

Q (100 years) = 30 m^3/s

Q (1000 years) = 40 m^3/s

Three cases have been considered:

Case 1. By 100 year flood with $Q_{design} = 30\ m^3/s$

◇ The principal gate open	$Q_{gate} = 22\ m^3/s$
◇ Overflow above weir crest	$Q_{weir} = 8\ m^3/s$
◇ Approx. upstream water level	200.85 m a.s.l.

Case 2. By 1000 year flood with $Q_{design} = 40\ m^3/s$

◇ The principal gate open	$Q_{gate} = 27\ m^3/s$
◇ Overflow above weir crest	$Q_{weir} = 13\ m^3/s$
◇ Approx. upstream water level	201.20 m a.s.l.

Case 3. Emergency case by a 1000 year flood

◇ Principal gate. Assumed closed (Not functioning or damaged)

◇ Sluice gate: Open	$Q_{SG} = 5\ m^3/s$
◇ Overflow above weir crest	$Q_{weir} = 21\ m^3/s$
◇ Overflow above principal gate	$Q_{PG} = 4\ m^3/s$
◇ Approx. upstream water level	201.50 m a.s.l.

The weir crest is assumed fixed on elevation 200.10 m a.s.l (see numeral 1). The foundation has been assumed to rest on a sound rock surface.

The left side of the weir is connected to a retaining wall (see numeral 2). The main gate opening has 2.70 m width by 2.40 m height, and the gate is driven with an electromotor. The sluice gate of 1.00 m by 1.00 m is also driven with an electromotor (see numeral 4 and 5).

The sumerged sill at el. 198.90 m a.s.l. (see numeral 6) allows to canalize the sediments into the sluice channel (see numeral 7) . The sluice channel has a slope of S = 0.08 which guaranties a supercritical flow through the sluice channel when the gate is open, carrying a maximum of sediments with a minimum of water losses.

A division wall or guiding wall (see numeral 8) helps to maintain the water flow in the correct direction.

The flood protection wall (see numeral 10) has been placed with its top on el. 201.90 m a.s.l. which shows that we still have a safety margin on the board of 0.40 m above the maximum possible water elevation of 201.50 m a.s.l. , That situation could occur in an emergency case when the main gate fails to open.

The retaining wall right (see numeral 12) conducts the flow into the intake. The trash rack has been designed with 5.30 m width and 1.25 m height and placed with an inclination of 75° with steel bars of 8 mm x 80 mm spaced 25 mm between bars. The rack is to be cleaned manually regularly avoiding at any time the rack to be clogged with floating materials.

An access and aeration pit of 0.70 m x 0.70 m (see numeral 14) allows inspections and control from the inside of the structure.

A water level control device foreseen for installation on the right retaining wall (see numeral 16), remoting measures at any time the water level in front of the intake structure.

After the entrance gate of the water intake, three rows of tranquilization racks are provided (see numeral 17) , the function of those racks is to force the coarse granular material carried by the flow to sink down , due to the tranquilization of the water flow into the desilting basin.

Footbridges for inspection and for service are provided at the entrance of the desilting basin (see numeral 18) and on top of the sluice gate and the principal gate (see numeral 9).

The desilting basin has been designed for a design discharge of $Q = 3.5$ m^3/s (see numeral 19). The main characteristics are:

◇ $L = 32.00$ m

- W = 4.00 m
- D = 3.50 m
- V = 0.25 m/s
- d = =.25 - 0.50 mm (desilting diameters)
- Bottom slope = 0.03
- Flushing gate = 1.00 m x 1.00 m (The details of the flushing gate are the same as the details given in the example of the water intake for Q = 2.6 m^3/s)

All the operations of the intake can be conducted in a centralized control room (see numeral 20) which can have a direct communication to the intake structure through a remote control cable (see numeral 21)

A low tension energy cable (see numeral 22) provide the necessary energy for the regular operation of all instruments and the mechanized gates. However the gates can be opened by hand in a case of emergency.

A small water dotation gate of 0.30 m by 0.40 m with manual operation (see numeral 23) allows to comply with water requirements for other purposes (water supply for example).

EXAMPLE OF A CONVENTIONAL WATER INTAKE WITH $Q = 3.5\ m^3/s$

LEGENDE

1. WEIR CREST El. 200.10 m a.s.l.
2. DELIMITATION OF THE ROCK SURFACE
3. LEFT SIDE RETAINING WALL
4. PRINCIPAL GATE OPENING 2.70 m x 2.40 m (ELECTRO POWERED)
5. SLUICE GATE 1.00 m x 1.00 m (ELECTRO POWERED)
6. SUBMERGED SILL ELEV. 198.90 m a.sl.
7. SLUICE CHANNEL S = 0.08
8. GUIDING WALL ELEV. 201.00 m a.sl.
9. GATES OPERATION PLATFORM ELEV. 201.90 m a.sl.
10. FLOOD PROTECTION WALL
11. RETENTION:
    - NORMAL LEVEL: 200.00 m a.s.l.
    - FLOOD CASE 1: 200.85 m a.s.l.
    - FLOOD CASE 2: 201.20 m a.s.l.
    - FLOOD CASE 3: 201.50 m a.s.l.
12. RETAINING WALL RIGHT
13. TRASH RACK 5.30 m x 1.25 m, inclined 75°
14. ACCESS AND AIREATION PIT 0.70 m x 0.70 m
15. ENTRANCE GATE 1.00 m x 1.00 m (WITH ELECTRO-CONTROL), USED ALSO AS FLOOD CONTROL.
16. WATER LEVEL CONTROL
17. TRANQUILIZATION RACKS
18. FOOTBRIBGE.
19. DESILTING BASIN FOR $Q = 3.5\ m^3/s$
    - L = 32.00 m
    - W = 4.00 m
    - D = 3.50 m
    - V = 0.25 m/s
    - d = 0.25 - 0.50 mm
    - Bottom slope = 0.03

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 3
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. / DESIGN BU / APP KRU	DRAWING NR 40093-3-1 APRIL 1987

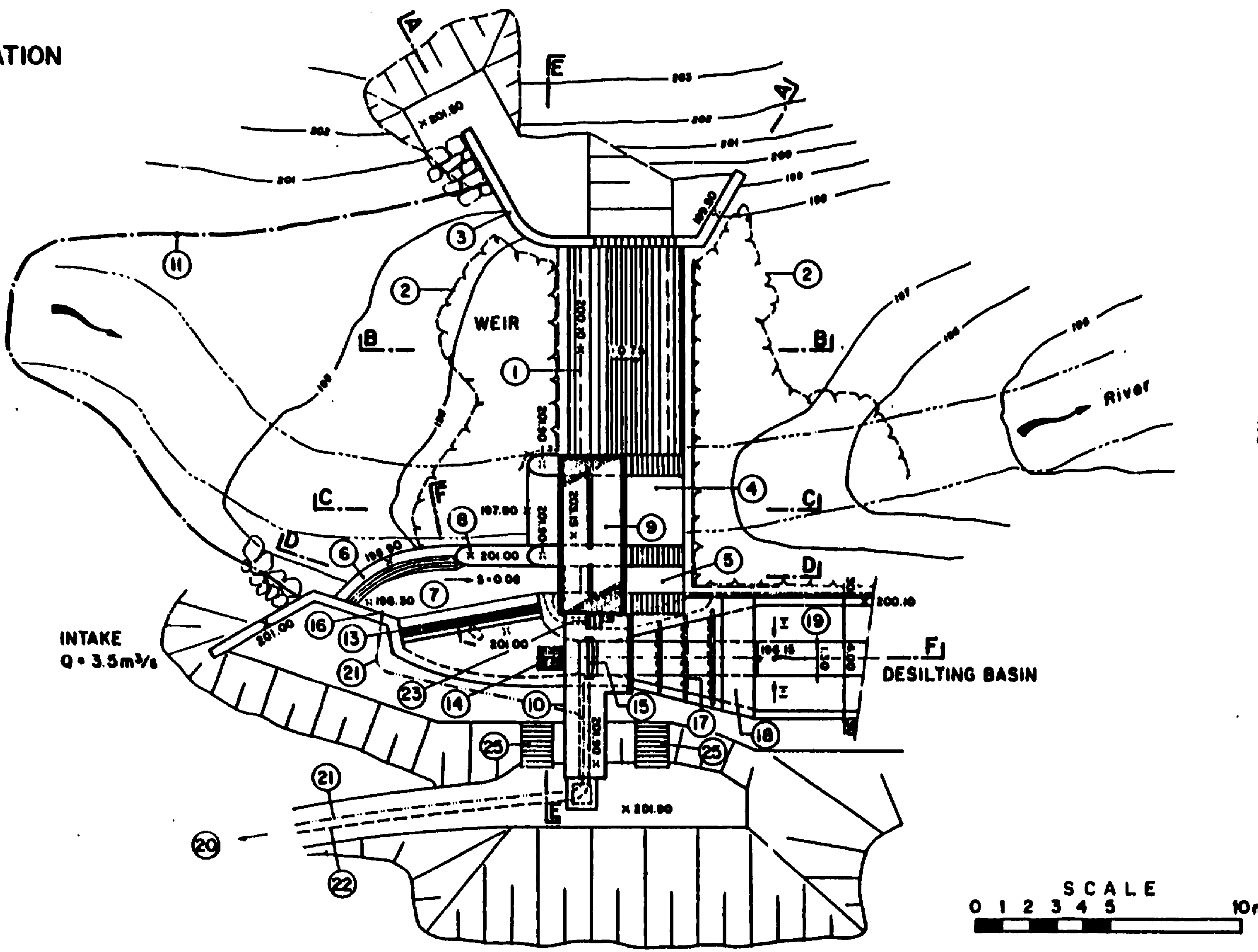
Flushing gate 1.00 m x 1.00 m. (For more details of desilting installation see example of desilting for Q = 2.6 m^3/s

20. OPERATION HOUSE
21. REMOTE CONTROL CABLE
22. LOW TENSION ENERGY CABLE
23. DOTATION GATE 0.30 m x 0.40 m WITH MANUAL OPERATION
24. CLOSURE WITH CLAY MATERIAL
25. STAIR

N.G.L. = NATURAL GROUND LINE
R. B. = RIVER BED

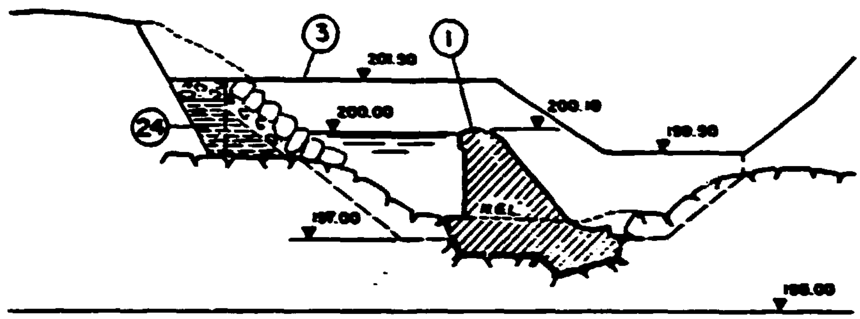
UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 3
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. DESIGN BU APP. HRU	DRAWING NR 40093-3-Ia APRIL 1987

SITUATION

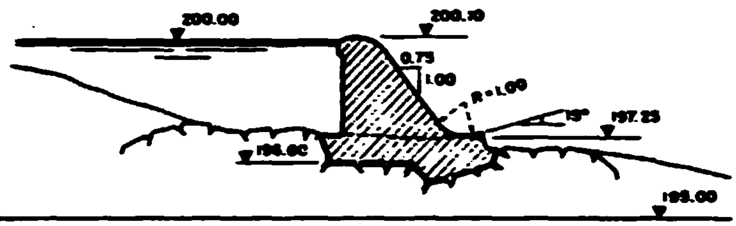


UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 3
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA.: BIE. DESIGN: BU APP.: RRU	DRAWING NR 40083-3-2 APRIL 1987

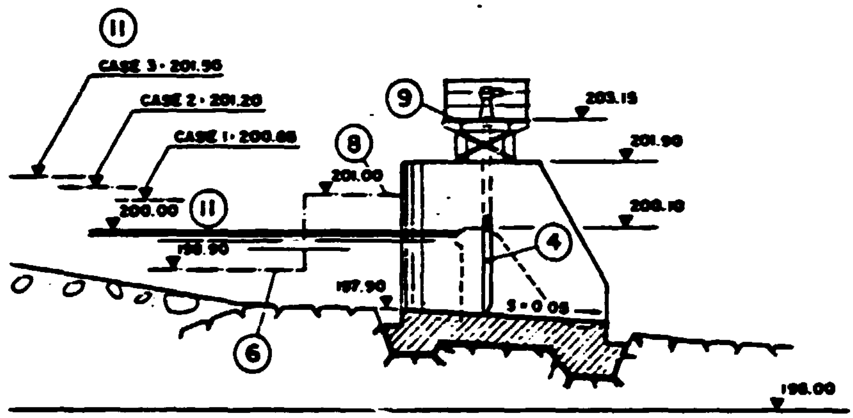
SECTION A - A



SECTION B - B

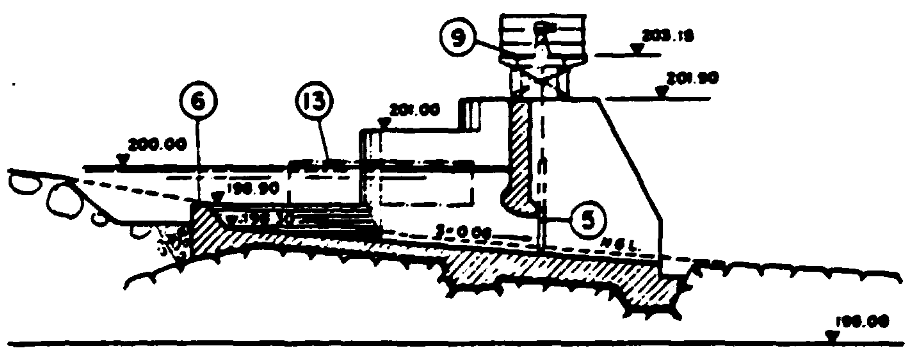


SECTION C - C



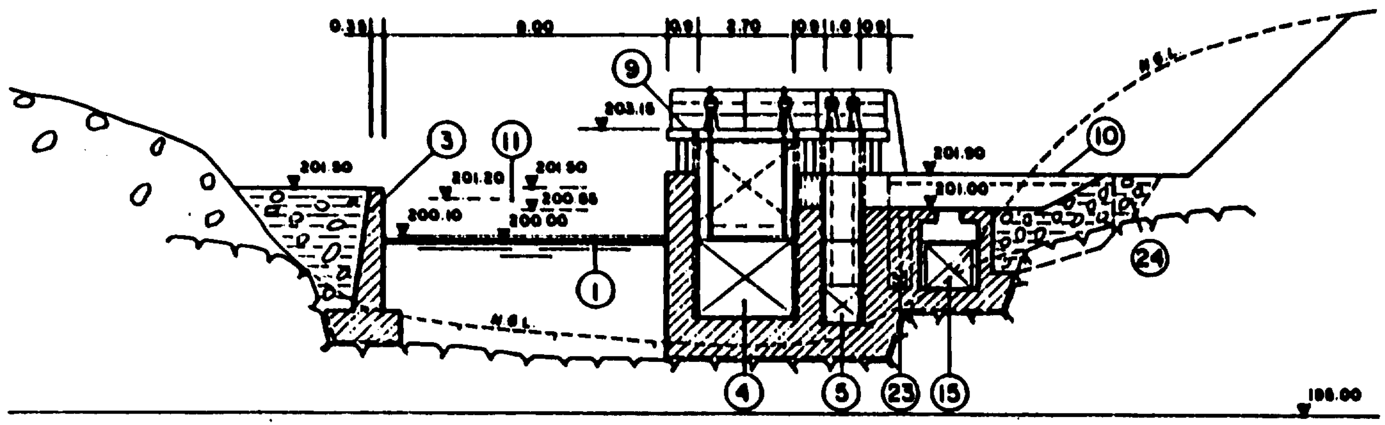
SECTION D - D

SCALE
0 1 2 3 4 5 10m

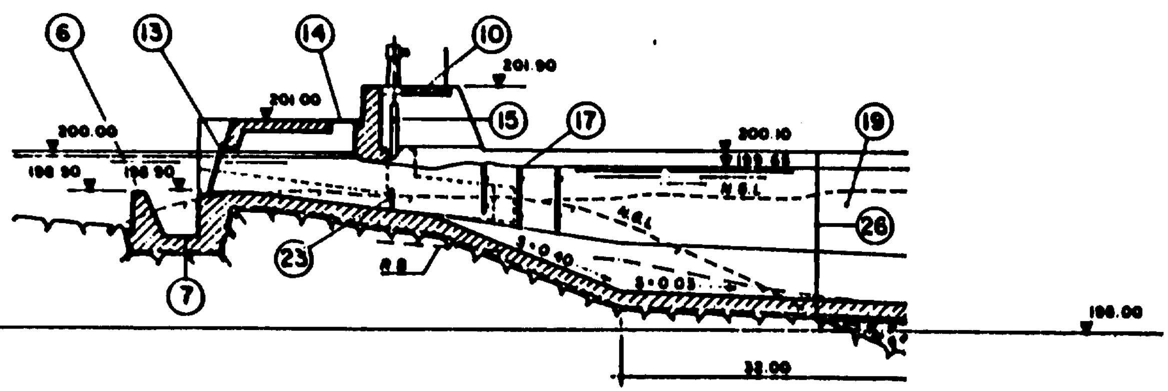


UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 3
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA.: SGR DESIGN: SU APP.: WRU	DRAWING NR 40093-3-3 APRIL 1987

SECTION E – E



SECTION F – F



SCALE

0 1 2 3 4 5 10m

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 3
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA.: SHL. DESIGN: SU APP.: KRU	DRAWING NR 40083-3-4 APRIL 1987

EXAMPLE 4

Drop Intake

with Desilting Basin

Q = 1 m3/s

EXAMPLE OF A DROP INTAKE FOR $Q = 1\ m^3/s$

An example of a Drop Intake for a $Q_{design} = 1\ m^3/s$ is presented in Drawing No.40093-4-1 and 4-2 This Example has been worked out taking into consideration the requirement of a minimum of maintenance.

The foundation of the structure has been assumed to be in a sound rock of a mountain creek.

Due to the fact that in mountain areas, little catchments can produce during floods much higher specific discharges per square kilometer than medium or large size catchment areas, the writers, for the purpose of this example with a Q_{design} of 1 m^3/s , have assumed 12 m^3/s for a 1000 year flood as an average taken from similar objects.

In a mountain area with a flood of 12 m^3/s a considerable amount of sediment transport sediments, is to be expected; it has to be separated from the water.

For a water intake with the above mentioned characteristics, an amount of 1 to 5 m^3 of sediments per day can be assumed, for discharges during floods of 4 to 6 m^3/s. Those values are taken from the experience made in other similar projects. This amount of sediments will require a minimum of one flushing a week and one flushing inmediatelly after every flood with discharges of the order of 4 - 6 m^3/s or more.

The inclination of the rack gives as 70% or an angle $\beta = 35°$, which ensures a better separation of clear water from the sediments with diameters greater than 20 mm and discharges of water that are less than 3.5 m^3/s. For larger discharges, other design consideration have to be made.

Only sediments with diameters of 0.5 to 20 mm will enter the desilting basin.

The intake has a small forebay excavated in the rock, which ensures a more regular and quieter approach of the water flow.

On the right side two openings of 0.80 m of width can be closed with wooden stop logs introduced into the lateral slots foreseen for this purpose.

The intake opening has a width of 1.80 m and is also provided with stop logs slots, to allow the closure of the intake entrance with two elements of 0.40 m each.

This type of intake require a weekly control and from time to time a cleaning of the rack to remove stones and leaves.

During a 1000 year flood the water will pass at about 0.70 to 0.80 m above the crest of the weir.

If there are any indication in the morphological features of the creek or river, that flash floods can occur, it is recommended to place the intake on a river side, and to prepare an upstream protection wall against the impact of the flood (this protection wall could be made with gabions).

The desilting plant will be operated manually about once a week and once after every big flood.

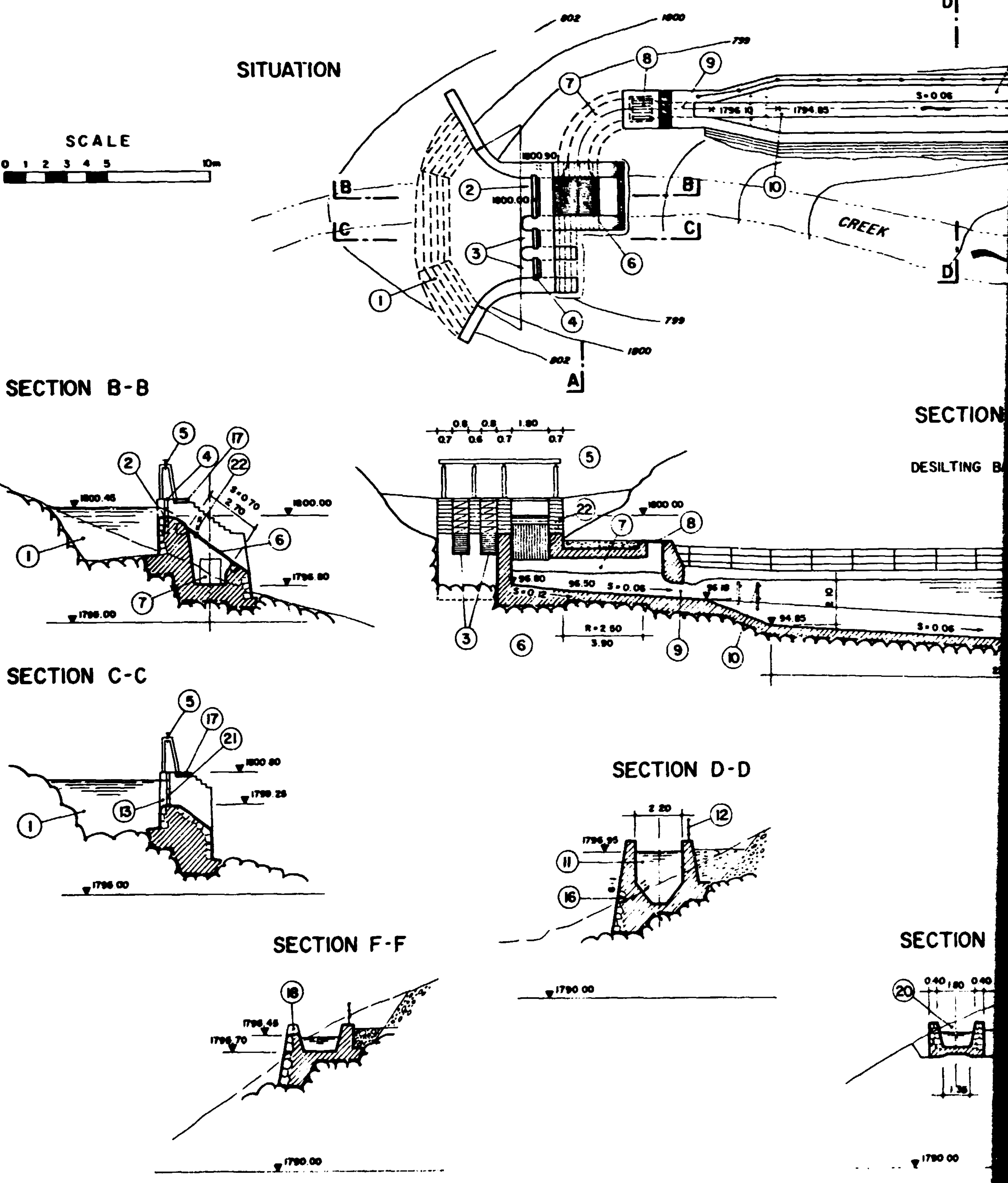
After the fall pit, below the rack, the water passes through the entrance canal and a throttled control section (see Drawing No. 40093-4-2, numeral 9), to control and to limit any excess of water flow during the flood, diverting the excedence of water through the lateral spillway constructed for this purpose (see numeral 18). At the entrance of the canal a flood protection wall is located (see numeral 19).

HIGH MOUNTAIN INTAKE AND DESILTING $Q = 1\ m^3/s$

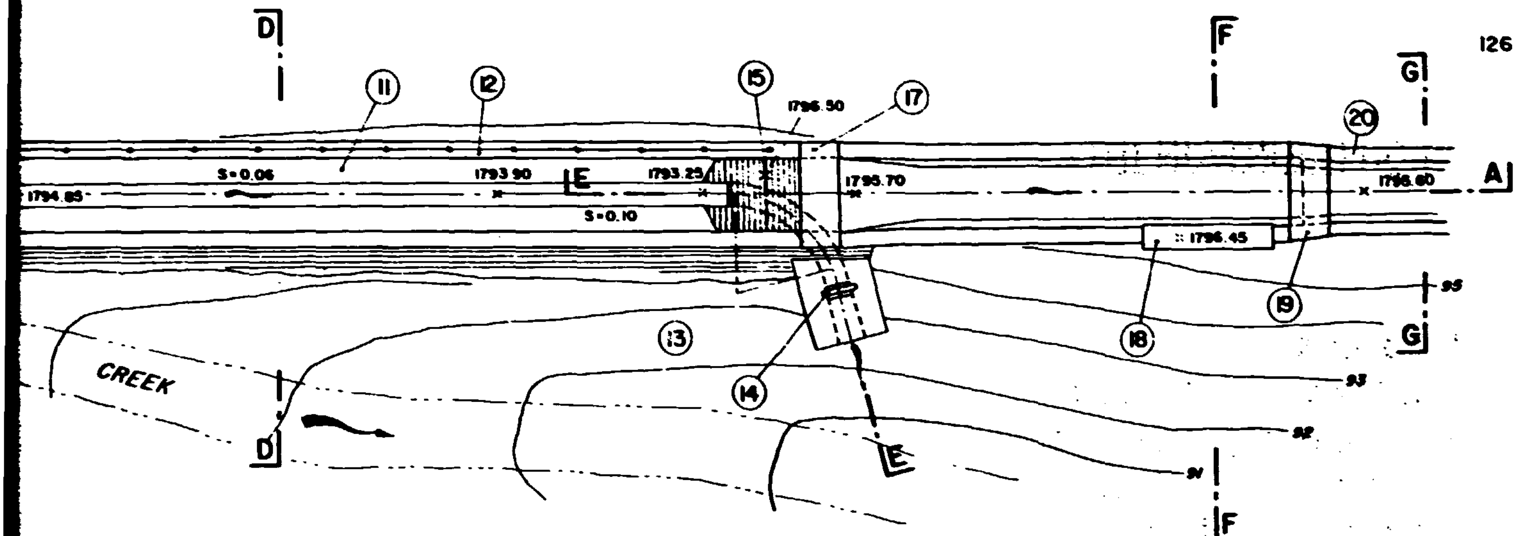
LEGENDE

1. Forebay
2. Intake sill elevation 1800.00 m a.s.l., W = 1.80 m
3. Temporary diversion, openning sill elev. 1799.25 m a.s.l., W = 0.80 m
4. Stop logs slots
5. Placing device of stop logs
6. Trash rack S = 0.70 m, bars Ø = 24 mm, space 25 mm
7. Entrance canal
8. Access and aeration pit
9. Control steel throtle 0.65/0.65 m
10. Tranquilization racks
11. Desilting basin, L = 22 m, W = 2.20 m, $Q = 1\ m^3/s$
    d = 0.3-2.0 mm, v = 0.25 m/s, W_0 =0.035 m/s
12. Handrail
13. Flushing channel
14. Flushing gate (manual) 0.80 /0.80 m
15. End spillway
16. Masonry
17. Passarelle
18. Flood Spillway
19. Passarelle and flood control
20. Adduction canal $Q = 1\ m^3/s$, S = 0.0012
21. Stop logs
22. Aeration

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 4
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. DESIGN BU APP. KRU	DRAWING NR 40093-4-1 APRIL 1987

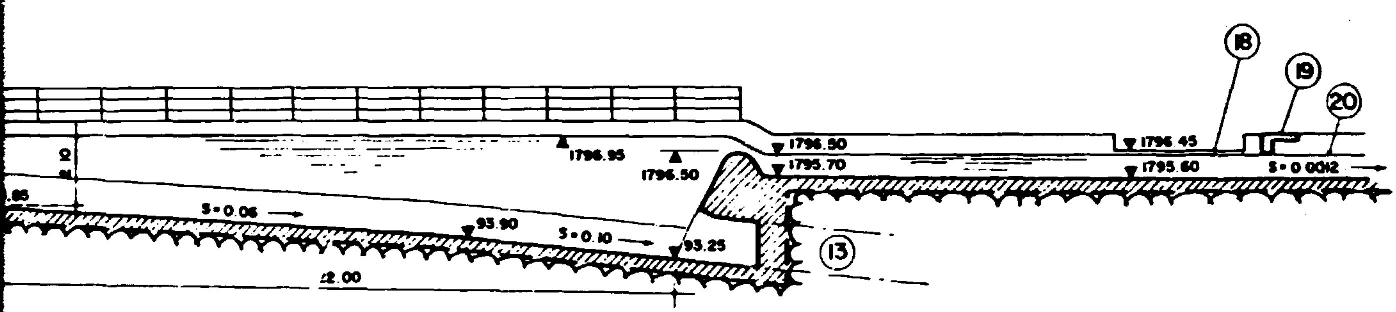

SITUATION
SCALE
0 1 2 3 4 5 10m
CREEK
SECTION B-B
SECTION C-C
SECTION D-D
SECTION F-F
SECTION
DESILTING B
1800.00
1800.90
1796.00
1790.00
S=0.06
S=0.12
R=2.50
3.90
2.20
0.40
1.80
1.36

SECTION 1

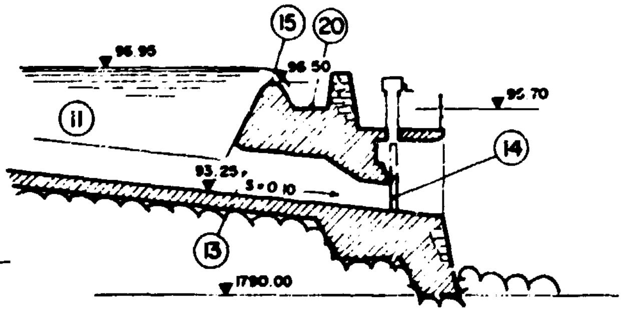


SECTION A-A

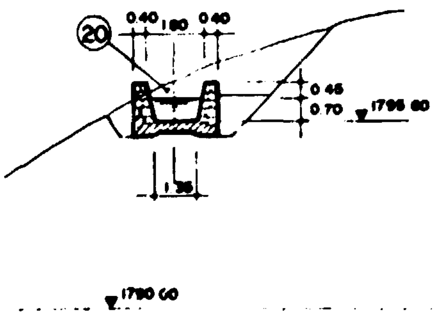
DESILTING BASIN $Q = 1\ m^3/s$



SECTION E-E



SECTION G-G



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 4
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. SGR DESIGN BU APP KRU	DRAWING NR 40093-4-2 APRIL 1987

EXAMPLE 5

Forebay

$Q = 1\ m^3/s$

EXAMPLE OF A FOREBAY FOR A Q = 1 m3/s

For the assumed example with a design discharge of Q = 1 m3/s, a forebay with a surface area of about 60 m2, and an average of 2.00 m depth with a regulation reserve of 0.50 m has been designed, see Drawings No. 40093-5-1 bis 5-3.

The minimum depth of 2.00 m has been calculated using Gordon's equation, which provides a minimum protection against the formation of vortices with the consequent danger of air entrainment

The equation of Gordon is:

$$d = 0.70 \times V \times \sqrt{D} \qquad (9.1)$$

where:

d = Depth of water (m)
v = Water velocity in penstock (m)
D = Diameter of Penstock (m).

With a Q = 1 m^3/s, v approximately equal to 3.50 m/s and a minimum penstock diameter of D = 0.60 m (which is the minimum diameter that allows an internal inspection) we obtain a d of about 1.9 m or in round figures, d = 2.00 m.

The minimum area of 60 m^2 is obtained from the experience with other similiar projects (see Table No. 9-1).

The above dimensions give a minimum water volume in the forebay of 120 to 150 m^3, with a regulation reserve of 0.50 m which is a safe value obtained from experience.

The entrance in the penstock is provided a trash rack, which for a discharge of Q = 1 m^3/s require a minimum gross area of 1.70 m^2 which gives a gross velocity of 0.59 m/s.

The trash rack is made of rectangular steel bars of 80 mm x 8 mm with separations between the bars of 25 mm, which allows a net area of approximately 1.30 m_2. (As a rule of thumb, the net area is approximately 3/4 of the gross area for this type of small intakes), therefore the net water velocity is of approx. 0.78 m/s.

The lateral emergency spillway has a regulation reserve of 0.50 m and an additional freeboard of 0.25 m as a minimum, which gives a possible overflow in an extreme situation of 0.75 m above the emergency spillway.

In the case of an overflow, and assuming that at the same time thatthe turbine is out of service (1 m^3/s) the canal will continue to bring water (another 1 m^3/s) and with a safety reserve of 0.5 x Q or 0.50 m^3/s, which makes a total of:

$$Q_{Spill} = 1 + 1 + 0.50 = 2.5\ m^3/s$$

assuming a depth of water on the spillway of 0.50 m , and knowing that:

$$Q_{Spill} = C\ \sqrt{2g}\ L\ H^{3/2}$$

or

$$L = Q_{Spill}\ /\ C\ \sqrt{2g}\ H^{3/2}$$

therefore L = 4.00 m , and using a Factor of Safety of 1.25 we obtain an

L = 5.00 m (length of the spillway)

The form of the spillway is calculated using the figure Nr. given previously, with an H of 0.50.m

For the calculation of the minimum water depth into the forebay we use again the Gordon's formula:

$$\mathbf{d_{min} = 0.70 \times V \times \sqrt{D}}$$

this time with a minimum design discharge of

Q = 0.50 m^3/s
D = 0.60 m (diameter of penstock)
F = 0.283 m^2 (area of penstock section)
V = 1.77 m/s (velocity of water in penstock)

$$d = 0.70 \times 1.77 \times \sqrt{0.60} = 0.96 = 1.00\ m$$

In the narrow valley used for this example, it is not possible to provide a compensation of water for more than 1/2 hour on top of the penstock, because the general configuration of nature does not allow this.

Only in cases in which a flat area is available on the site, would it be possible to provide a reserve volume in the forebay for 1 - 1.1/2 hours of peak energy.

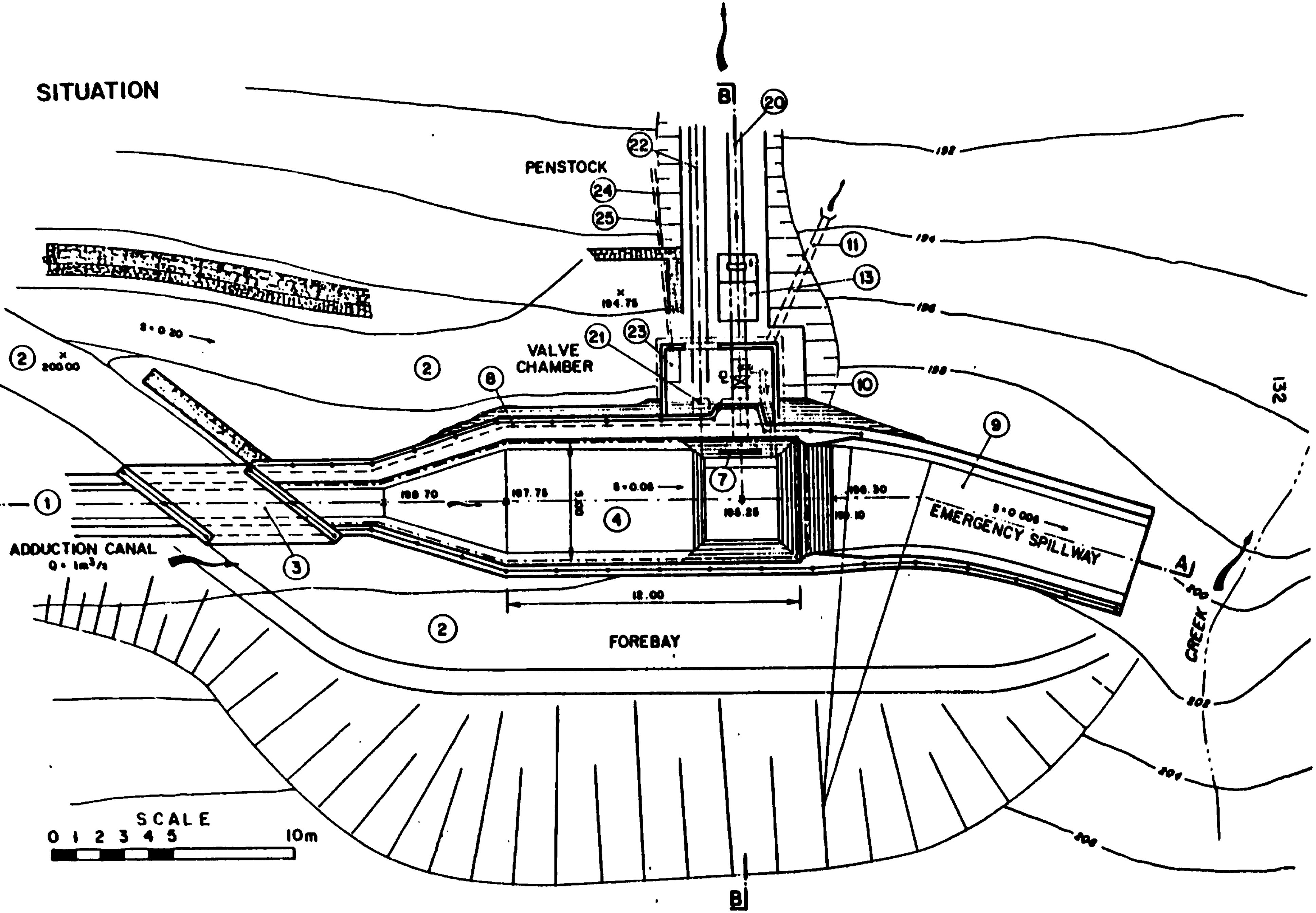
In some isolated plants, energy is provided only during certain hours. without consumption in other hours, allowing to increase the production of energy, if enough pondage volume could be made available in the forebay or in another type of reserve configuration, allowed by the local topographic conditions.

FOREBAY FOR $Q = 1\ m^3/s$

LEGENDE

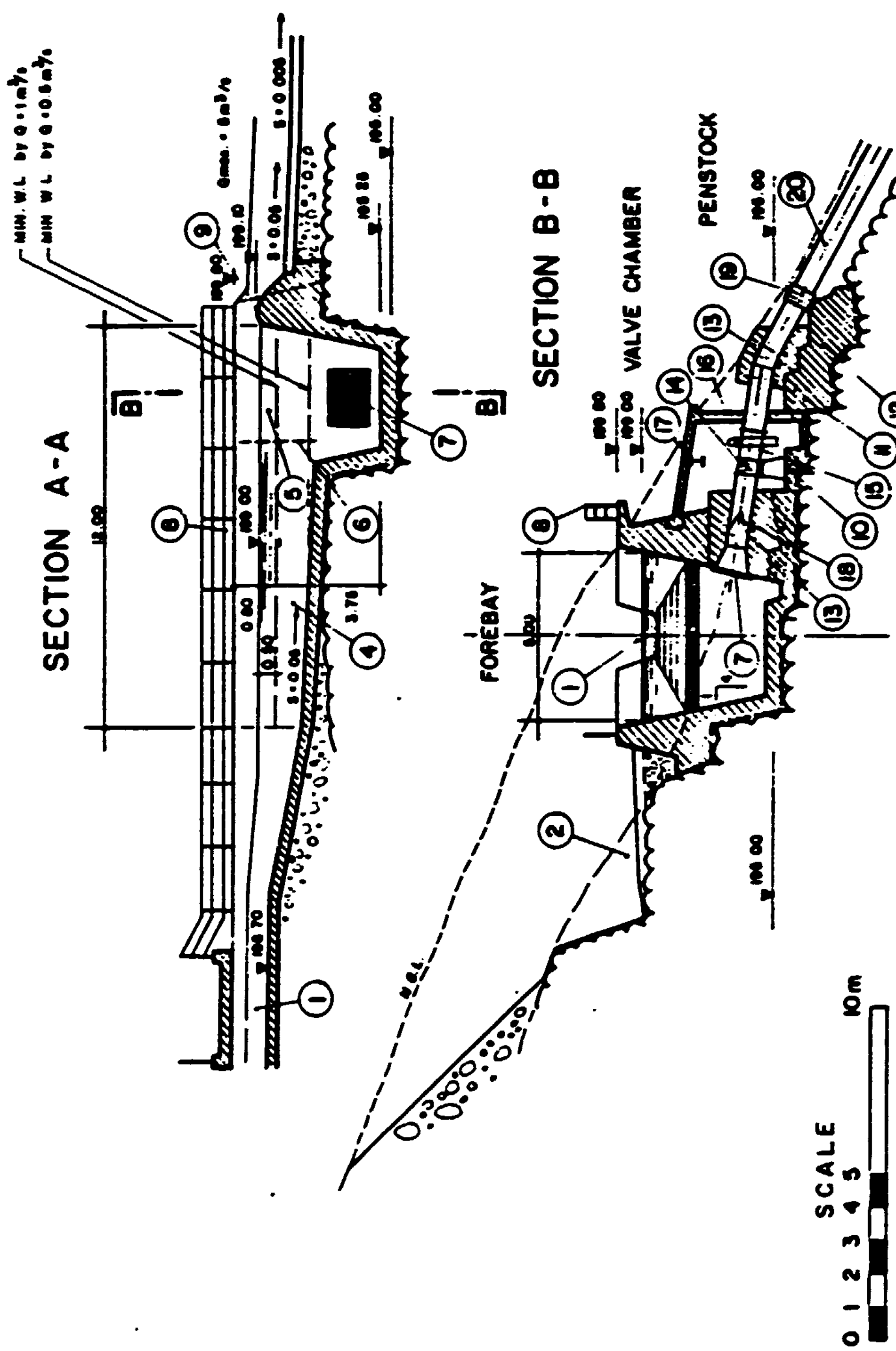
1. ADDUCTION CANAL $Q = 1\ m^3/s$
2. ACCESS ROAD
3. BRIDGE
4. FOREBAY
5. REGULATION VOLUME, SURFACE ELEV. 199.00 m a.s.l. 5m.x 12 m
6. WATER LEVEL CONTROL
7. INTAKE TRASH RACK 1.70 m x 1.00 m
8. PASSAGE WITH HANDRAIL
9. EMERGENCY SPILLWAY
10. SLUICE VALVE D = 200 mm
11. EVACUATION PIPE D = 400 mm
12. ANCHOR BLOCK FOUNDATION
13. SECOND CONCRETE
14. AUTOMATIC BUTTERFLY VALVE D = 500 mm
15. DISMOUNT PIPE AND FLANGE
16. AIR VALVE (RELEASE AND ENTER)
17. RAIL FOR ERECTION
18. STEEL TRANSITION
19. EXPANSION
20. STEEL PIPE D= 200 mm
21. WINCH
22. TRACK (FUNICULAR)
23. ELECTRIC AND REMOTE CONTROL
24. ELECTRIC CABLE
25. REMOTE CABLE

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 5
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA.: DESIGN: BU APP.: KRU	DRAWING NR 40093-5-1 APRIL 1987



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH

EXAMPLE 5
DRAWING NR 40083-5-2
APRIL 1987



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 5
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA.: BIEL DESIGN: BU APR.: HRU	DRAWING NR 40093-5-3 APRIL 1987

EXAMPLE 6

Forebay

$Q = 2.6\ m^3/s$

EXAMPLE OF A FOREBAY FOR A Q = 2.60 m^3/s

An example of a forebay for a design discharge of 2.60 m^3/s is presented in Drawing No. 40093-6-2.

Characteristics: Intake type with max. slide gate of 950 x 950 mm, with electrical operation, with water filled gate shaft and side emergency overflow. The gate shuts down automatically at a velocity excess in the penstock; the gate can also be closed from the powerhouse. This solution presents an easy erection as well as control and repair.

The dimensioning of the main parts has been made in a similar way as for the forebay for Q = 1 m^3/s presented in the previous example.

The adduction canal (see numeral 1) has been calculated for a minimum discharge of 2.60 m^3/s that enters the forebay at the elevation 198.15 m a.s.l. (see numeral 3), The water surface cover an area of 5.80 m of width by a length of 18.00 m and the bay has a reserve margin for water oscillations of 0.75 m.

The intake trash rack has been dimensioned 1.80 m in height by 2.40 m length, with its center at elevation 194.50 m a.s.l.

The emergency spillway has been placed with its sill at elevation 197.25 (see numeral 7)

A sluice valve of 250 mm of diameter (see numeral 8) has been foreseen for a manual operation. This valve allows a periodical cleaning of the forebay.

The complete foundation of the forebay will be made with a reinforced concrete of the type PC-300 (see numeral 10).

The head slide gate 950/950 mm (see numeral 11) is reached from the trash rack through the intake mouth which will be electrically operated from the platform at elevation 199.35 m a.s.l.

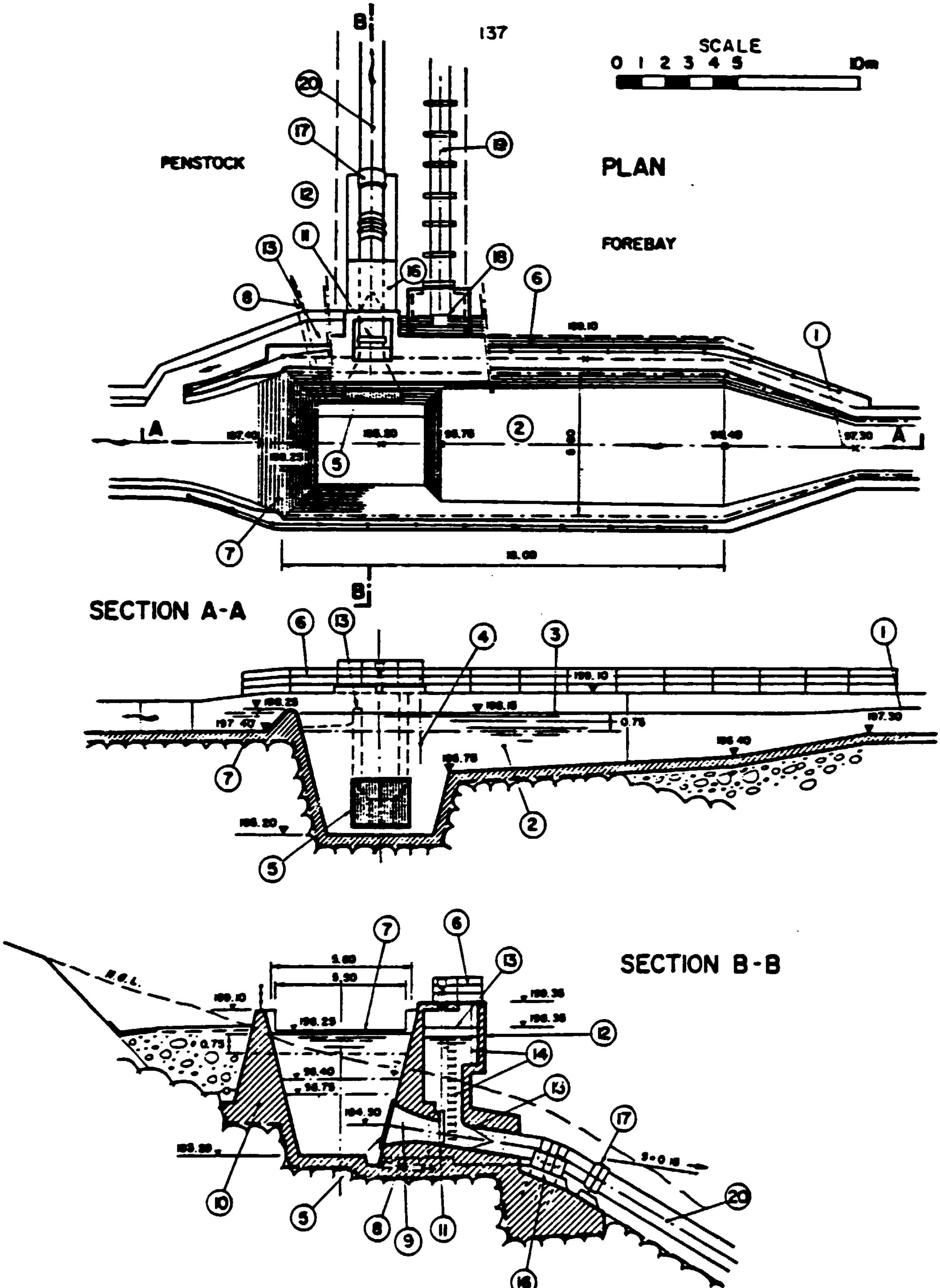
The gate shaft (see numeral 12) serves as aeration of the penstock and in the case shutdown of the turbine; the water can pass through the emergency overflow (see numeral 13). The penstock is designed with a diameter of 950 mm (see numeral 20).

FOREBAY FOR Q = 2.6 m³/s

LEGENDE

1. ADDUCTION CANAL Q = 2.60 m^3/s
2. FOREBAY
3. FOREBAY OPERATION VOLUME WITH WATER LEVEL AT ELEV. 198.15 m a.sl., 5.80 m OF WIDTH AND 18.00 m LENGTH, WITH 0.75 m FOR POSSIBLE WATER OSCILLATIONS.
4. WATER LEVEL CONTROL
5. INTAKE TRASH RACK 1.80 m x 2.40 m
6. FOOTBRIDGE WITH HANDRAIL
7. EMERGENCY SPILLWAY CREST ELEV. 198.25 m a.s.l.
8. SLUICE VALVE D = 250 mm
9. INLET
10. REINFORCED CONCRETE PC-300
11. HEAD GATE 950/950 mm
12. GATE SHAFT (FOR ACCESS AERATION AND OVERFLOW)
13. OVERFLOW WITH EVACUATION CANAL
14. LADDERS
15. BEGINN OF STEEL CONDUIT
16. ANCHOR BLOCK A-1
17. EXPANSION JOINT
18. WINCH HOUSE
19. FUNICULAR
20. PENSTOCK D = 950 mm .

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 6
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA · DESIGN BU APP : WHU	DRAWING NR 40003-6-1 APRIL 1987



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 6
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. : DESIGN: BU APP. : KRU	DRAWING NR 40093-6-2 APRIL 1987

EXAMPLE 7

GENERAL LAYOUT OF

HYDROPLANT AND PENSTOCK

$Q = 1\ m^3/s$

EXAMPLE OF A GENERAL LAYOUT OF A HYDROPLANT AND A PENSTOCK WITH Q = 1 m^3/s

The general layout with a longitudinal profile of a hydropower plant is presented in Drawing No. 40093-7-1; it represents a complete installation for a small power plant for a Q = 1 m^3/s.

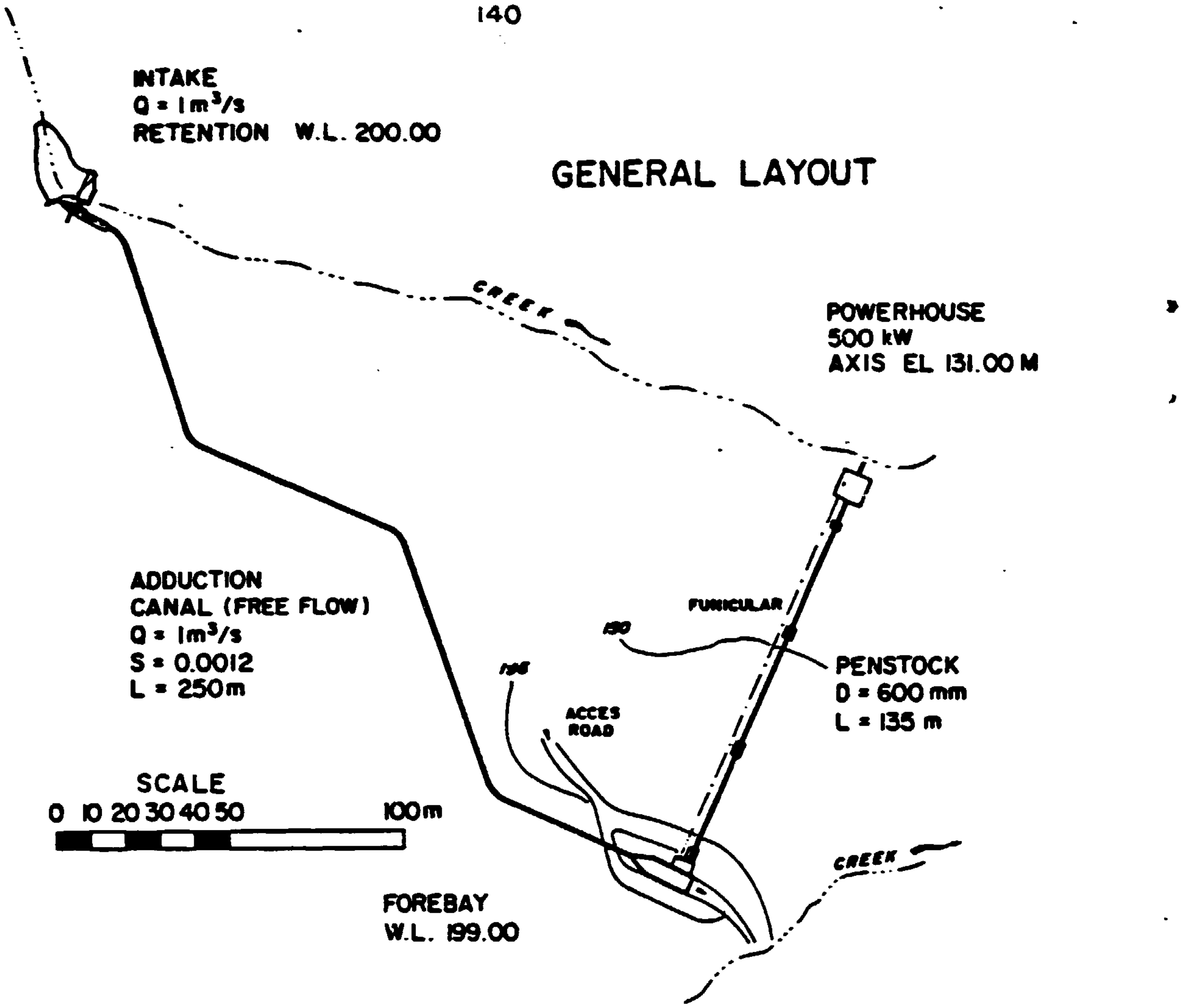
The choice of the intake and powerhouse location receives first importance. The shortest ratio distance/gross height must be as small as possible.

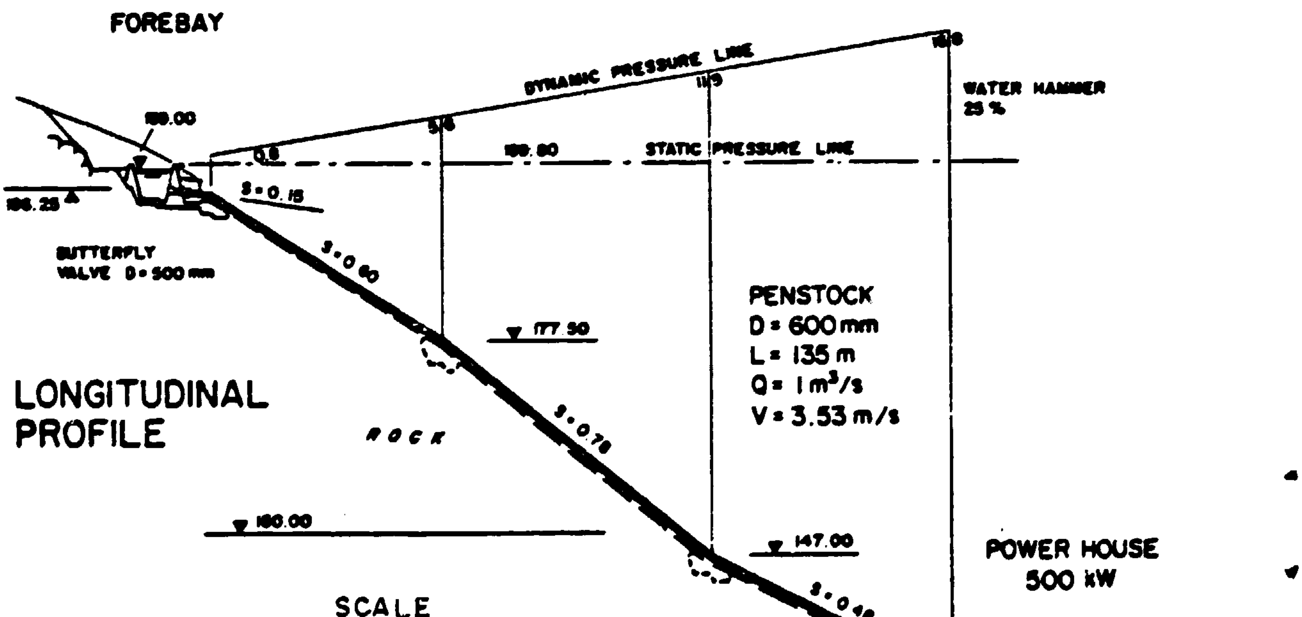
The location of the intake requires a particular attention; it will be placed at the highest possible elevation, taking into account the location in a bend of the river, under the best possible geological conditions, and a narrow cross section. The adduction canal should be able to flow from there with a favourable tracing topographically and geologically

This general layout assumes the retention and the intake structure at the elevation 200.00 m a.s.l. The water is then conveyed through an adduction canal (with free flow) to the forebay at elevation 199.00 m a.s.l. and from there to the powerhouse through the penstock of 135 m of length and a diameter of 600 mm

A powerhouse with an installed capacity of 500 kW is placed at the elevation 131.00 m a.s.l.

A dynamic pressure of 25 % of the total static pressure has been added, to allow for possible water hammer effects (it is to be remember here that in all cases the dynamic effect should be computed and it should be ascertained if a surge chamber is required or not) . The velocity of the water inside the penstock is 3.53 m/s, which can be considered as safe. The penstock line between the forebay and the powerhouse should be as short as possible with the most favourable ratio length/height in order to obtain the maximum possible velocity of the water in the penstock.





UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZ...			EXAMPLE 7
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. DESIGN APP.	BU KRU	DRAWING NR 40093-7-1 APRIL 1987

EXAMPLE 8

PENSTOCK

$Q = 2.6\ m^3/s$

EXAMPLE OF A PENSTOCK FOR Q = 2.6 m^3/s

An example of a penstock for a Q = 2.6 m3/s is presented in Drawing No. 40093-8-1.

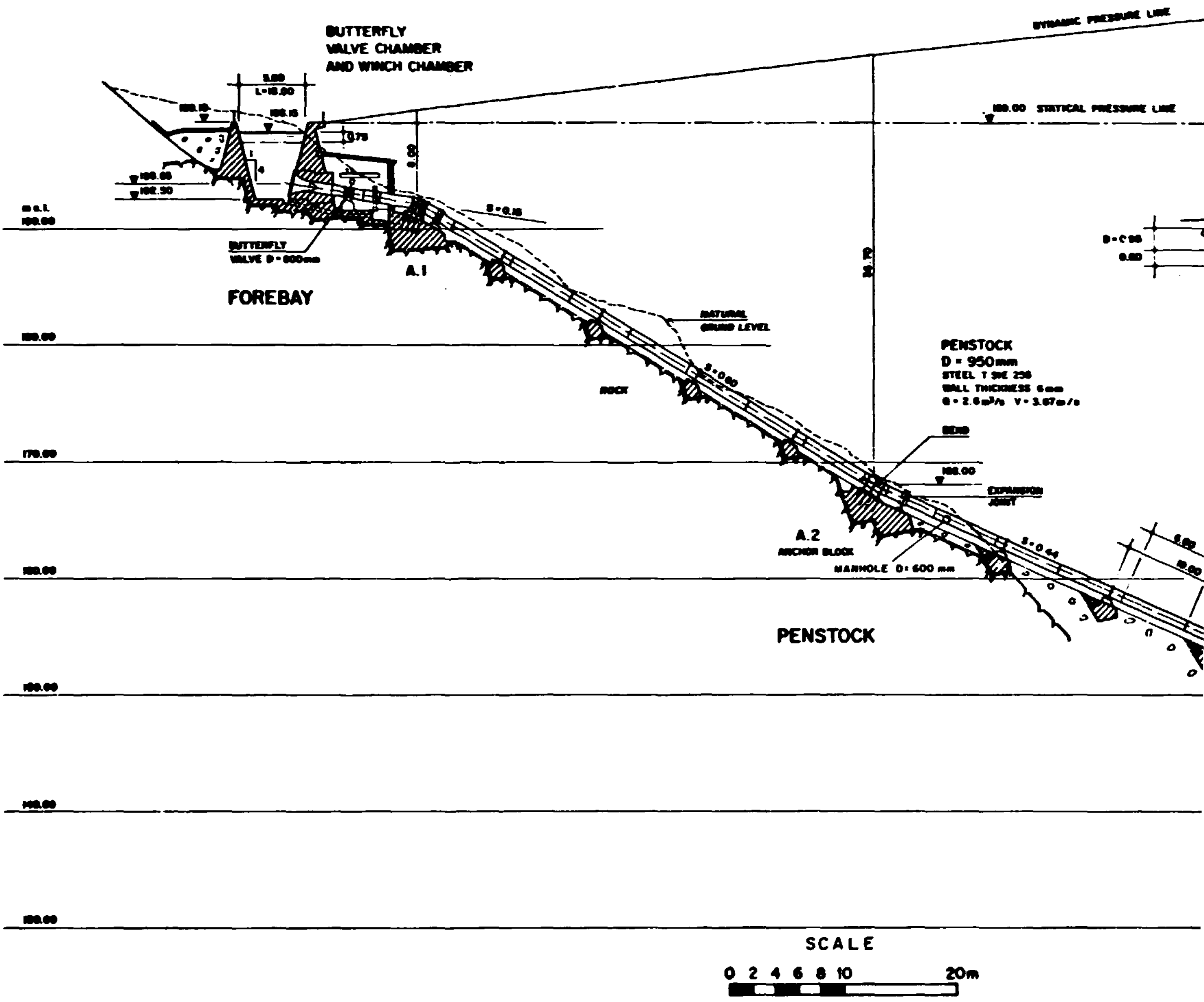
The first part of the penstock is assumed to be installed in a sound rock foundation, the second part is assumed in a well consolidated alluvial soil.

A forebay with a butterfly valve with a diameter of 800 mm gives control a the entrance of the penstock. It is possible to place a slide gate instead of the butterfly valve.

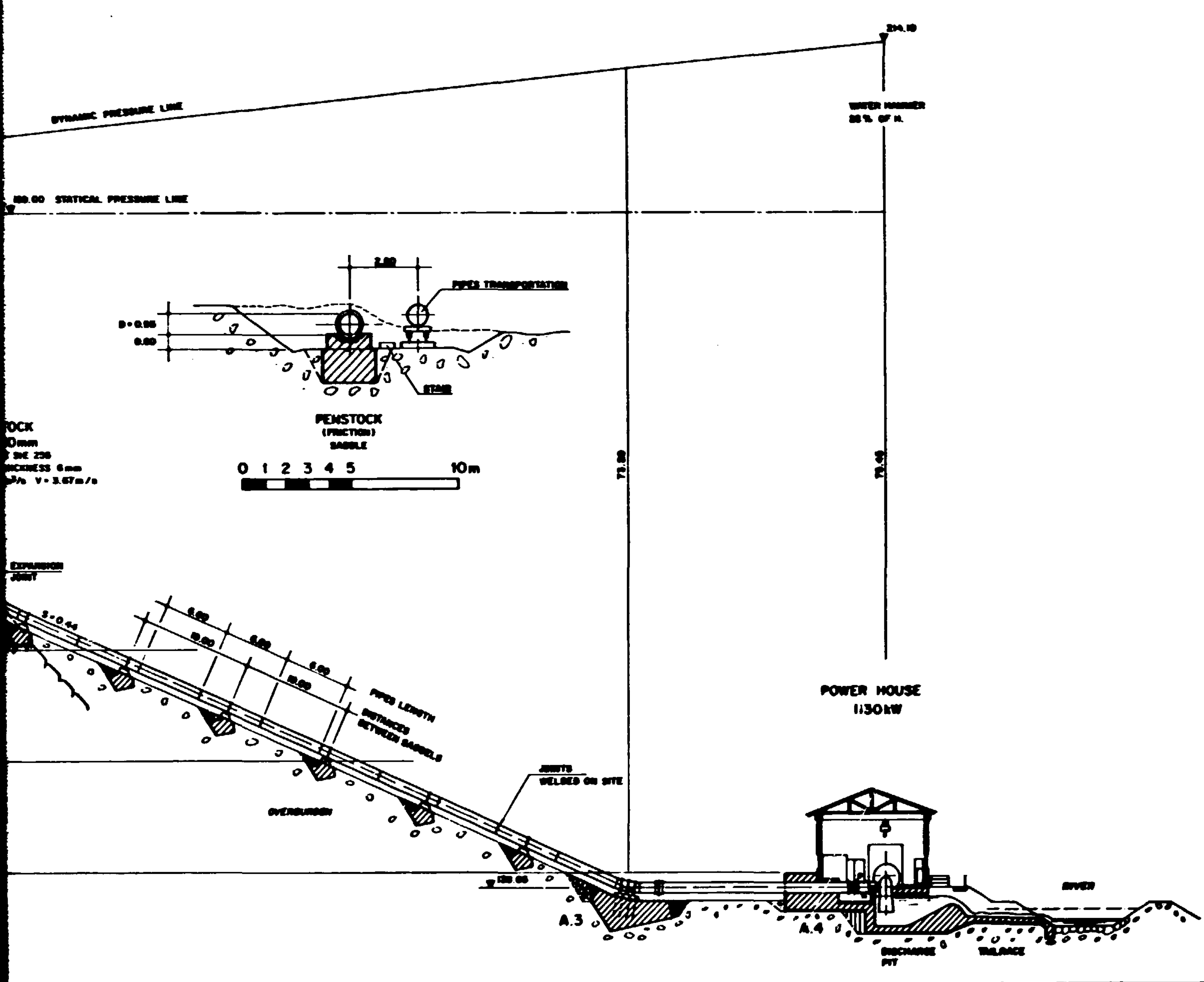
Two types of anchors blocks are presented, one for the upper part,which founded in rock represents the most favourable condition for a penstock and the second for the lower part founded in overburden of a well consolidated alluvial soil. This solution is only possible if the geological conditions of the natural soil are stable enough. If it is not possible to find a foundation in rock, and the saddles are to be founded in the overburden: a slope of S = 0.44 is a maximum to be allowed.

When no rock foundation is possible the pipe must be completely embedded in concrete to make a low repartition pressure on the overburden, with a slope of the total length of no more than S = 0.60.

The saddles are placed at 10. m intervals, the penstock has a diameter of 950 mm with 6 mm steel thickness. The penstock is assumed welded on site, each 6. m. The water velocity in the penstock is 3.67 m/s.


BUTTERFLY
VALVE CHAMBER
AND WINCH CHAMBER
DYNAMIC PRESSURE LINE
STATICAL PRESSURE LINE
BUTTERFLY
VALVE D = 600mm
A.1
FOREBAY
NATURAL
GRUND LEVEL
ROCK
PENSTOCK
D = 950mm
STEEL
WALL THICKNESS 6mm
BEND
EXPANSION
JOINT
A.2
ANCHOR BLOCK
MANHOLE D: 600 mm
PENSTOCK
170.00
SCALE
0 2 4 6 8 10 20m

SECTION 1



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 8
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. ZUA DESIGN SU APP KRU	DRAWING NR 40093-8-1 APRIL 1987

SECTION 2

EXAMPLE 9

POWERHOUSE

$Q = 2.5$ m^3/s

EXAMPLE OF A POWERHOUSE WITH Q = 2.6 m^3/s

An example of a powerhouse with a Q = 2.6 m3/s and an installed capacity of 1130 kW is presented in Drawings No. 40093-9-1 / 40093-9-3.

A turbine of the Francis type has been selected for this scheme.

The powerhouse present the main classical elements required for small and mini hydropower plants.

Enough free room has been left available for the maintenance and operation of the power plant.

For other types of turbines, the discharge pit as well as the tail race channel should be adapted according the requirements of the turbines.

The power house should always be located in a protected area near the river, the tailrace must be placed high enough above the level of a 1000 years flood, and need an efficient protection against floods.

The access to the powerhouse should be open all year round.

The foundation slab should be made with reinforced concrete PC 300; the stability of the slab must be checked considering the maximum pressure at the butterfly valve during a shutdown, checking the pressure on the natural ground.

The slab serves for suporting the equipment as well as a mass to dampen any possible vibrations during operation of the machinery.

The structure of the powerhouse must protect the equipment and instruments from the weather.

The superstructure can be built with concrete or masonry blocks walls (min. 30 cm thick); if the walls are constructed with cobbles masonry, the thickness should be of a minimum of 50 cm.

The position of the hook of the crane must be high enough according to the indications of the equipment suppliers (for assembling and disassembling).

The roof is made of transversal steel beams with a steel plate for bearing, to allow for the suspension of the crane beam.

The roof structure can be made in wood or in steel (with enough contravent for the wind action).

The cover of the roof can be made, if possible, of wooden planks ceiling, fiberglass insulation or corrugated metal sheets.

The bottom floor of the powerhouse should present a clean surface, the cable channels should be protected with steel covers.

The access door has to be designed large enough to permit the passage of the largest pieces of equipment.

The powerhouse in general should have an adequate drainage system.

Adequate ventilation of the powerhouse should be provided with roof vents or other similar elements.

The Tailrace should be as short as possible, the overflow has to be designed according with the specification of the suppliers.

The stop logs slots should be made on the concrete walls to allow for future possible stop logs installation.

A good foundation is required at the connection with the river, and a protection, if necessary, with rip-rap.

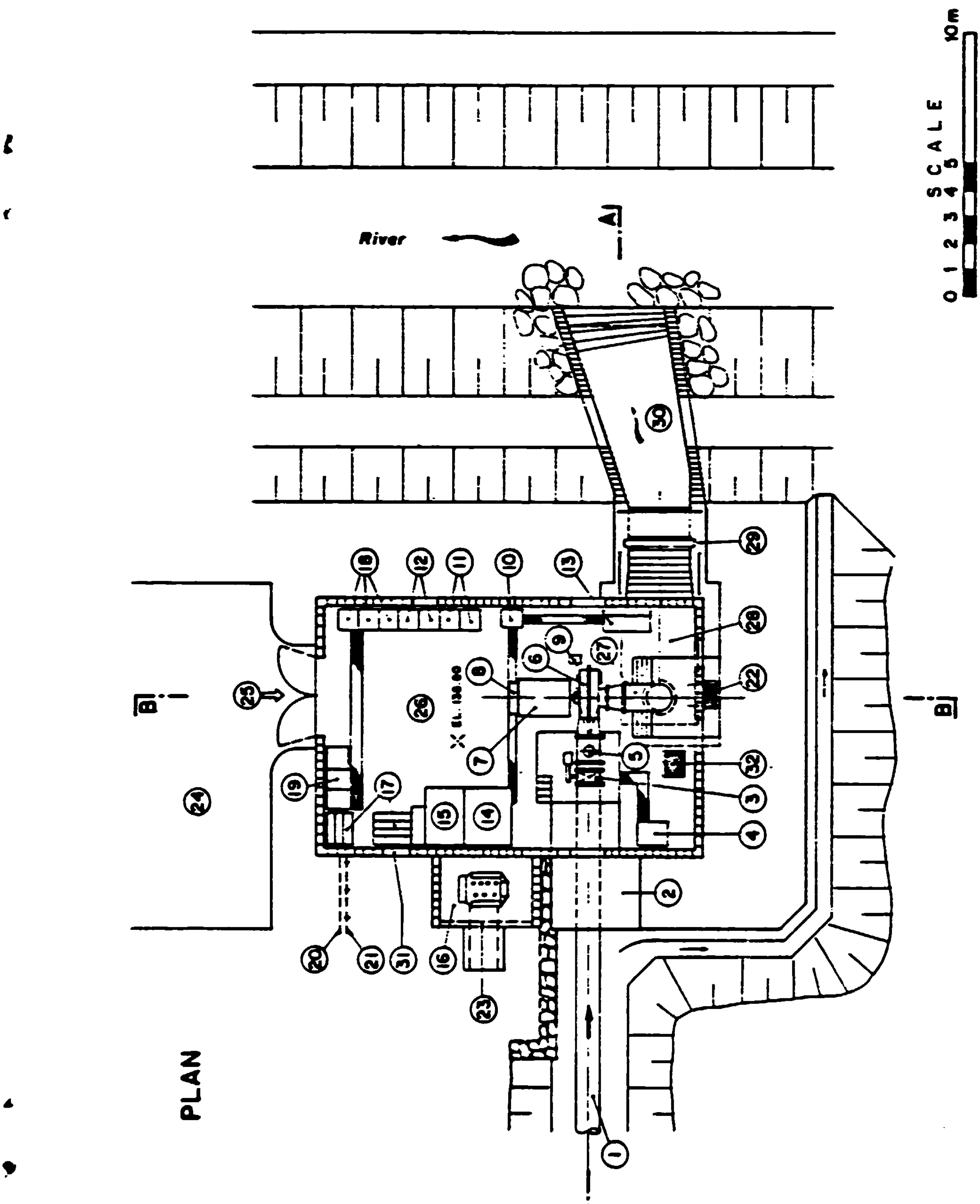
THE POWERHOUSE FOR Q = 2.6 m^3/s

1. Penstock
2. Anchor block
3. Butterfly valve D = 900 mm
4. Oil pressure unit
5. Manhole
6. Turbine
7. Generator
8. Fly wheel
9. Governor
10. Generator neutral cubicle
11. Start and stop sequence
12. Exitation
13. MCC
14. Generator switchyard
15. Service transformer
16. Main transformer
17. Batteries
18. Light-power DC
19. Remote control of waterway
20. Cable for low tension
21. Remote control cable
22. Aeration of the tailrace
23. Fence door
24. Access
25. Access door
26. Erection area
27. Draft tube
28. Discharge pit
29. Stop logs groove
30. Tailrace
31. Window and ventilation
32. Dewatering pit

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 9
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA.: DESIGN: BU APP.: NRU	DRAWING NR 40093-9-1 APRIL 1987

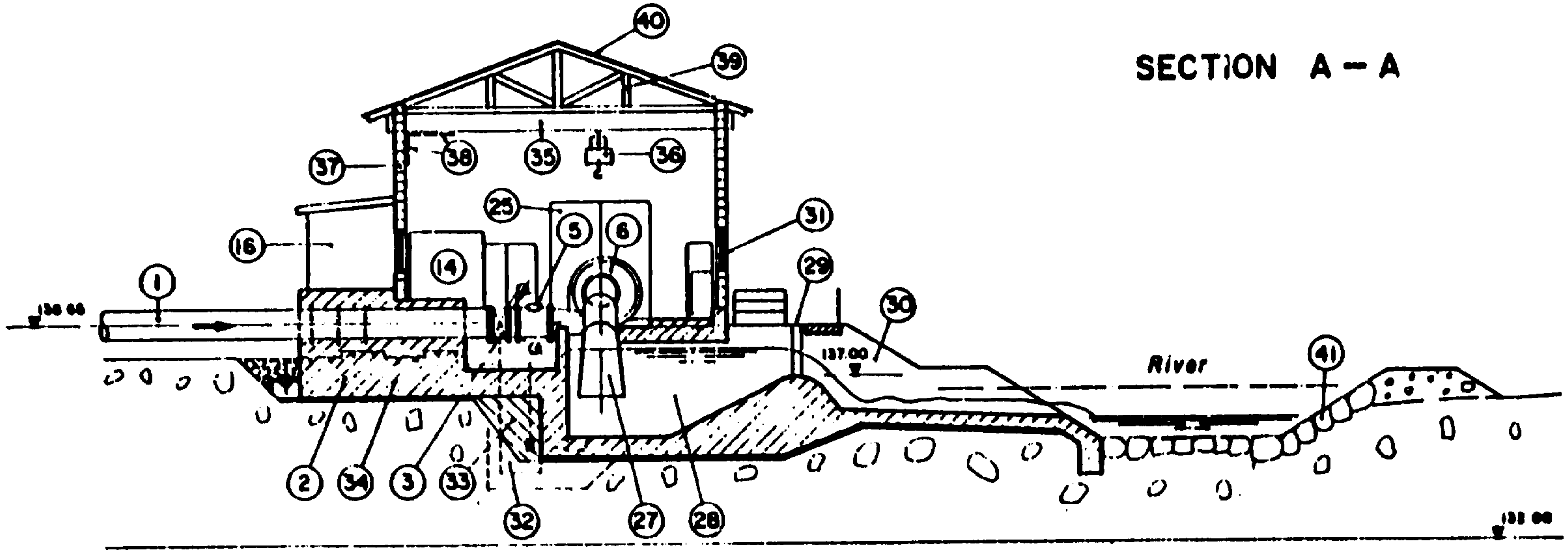
33. Lean concrete PC 200
34. Reinforced concrete PC 300
35. Steel beams
36. Cran 6 ton.
37. Concrete masonry blocks
38. Possible noise insulation
39. Timber work or steel work
40. Roof: corrugated metal sheets with fiberglass insulation below
41 Rip-rap

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 9
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. DESIGN: BU APP. KRU	DRAWING NR 40093-9-1a APRIL 1987

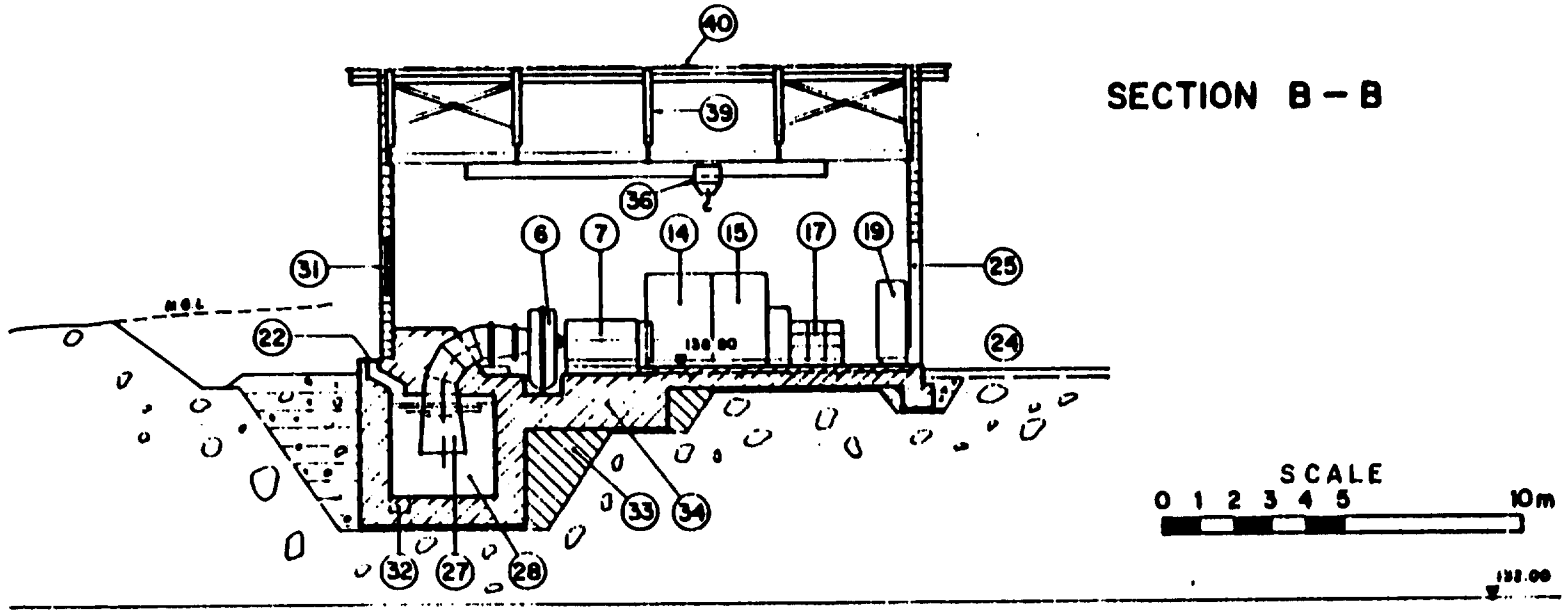


UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 9
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH	DRA. DESIGN. SU APP. KRU	DRAWING NR 40093-9-2 APRIL 1987

SECTION A — A



SECTION B — B



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION	EXAMPLE 9
ELECTROWATT ENGINEERING SERVICES LTD. ZURICH — DRA.: DESIGN: BU APP.: NRU	DRAWING NR 40093-9-3 APRIL 1987