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REPORT ON STANDARDIZATION OF CIVIL WORKS FOR SMALL HYDROPOWER PLANTS

Engineers + Consultants

Zurich - Switzerland
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前人種樹，後人乘涼。

*'Trees planted in the past
cast their shadow today'*
(Chinese proverb)

FOREWORD

The following manual has been prepared at the suggestion of UNIDO, because of the obvious lack of literature in the field of Small Hydropower. Only in recent years has Small Hydropower gained in importance, especially in developing countries. In such countries plants of this type are of great usefulness in remote areas where they can help speeding up development.

It is often thought that Small Hydropower Plants can be built by indigenous means by the local population; however, this requires that manuals are written in a way which is understandable and accessible to those people.

This manual is not a substitution of sound, well-established design practice, however, it points out those solutions and methods which are thought to be more easily applicable under the given circumstances. It also discusses common design errors which are often encountered with facilities built in the past. Faulty design can often produce considerable economic losses and in some cases even lead to total failure.

This manual has been prepared by the following engineers: A. Krumdieck (hydraulic structures), E. Bucher (intakes), R. Lützelschwab (topography), J.P. Gisiger (geology), Dr. F. Laufer (hydrology), and Messrs. E. Bernhard and J. Litscher with a valuable advice. The authors gratefully acknowledge the support given by Electrowatt Engineering Services Ltd. for making available their data base on hydraulic projects. The authors also wish to thank UNIDO for their interest in sponsoring this project.

It is hoped that the manual will help to promote the construction of Small Hydropower schemes and become a standard reference in the literature concerning this field of energy development.

Alfonso Krumdieck
Civil Engineer

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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, utilizing as far as possible their own resources.

Recognizing that the main problems impeding 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 considerable experience - in designing, realizing and operating - obtained in countries in which hydroelectric power generation has traditionally been one of the main sources of energy over periods of many years.

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

◊ savings in non-renewable energy sources

- easier and simplified realization

- minimum design costs

- realization by local and possible less highly trained personnel

- maximizing the use of locally obtained components (civil, mechanical and electrical)

- enabling technological transfer

- reducing expenditure of foreign currency, in particular in relation to import of combustibles

- 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 FOR SMALL HYDROELECTRIC POWER SCHEMES

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

Not-withstanding this, the aim of the present study is to provide a general guidance in regard to the design and realization of the most important components of such plants; in other words, to provide basical calculations, sketches and drawings, which will be easy to interpret and adapt, even if the data of the actual plant to be realized are somewhat different.

In attempting to deal with this type of approach, we have divided a normal hydropower scheme into its most useful components, and have endeavoured to develop each one of those components into independent items, that should, within certain reasonable limits, permit to unify 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 children use to construct the most varied structures with the same basic elements).

By following this "Lego System" philosophy of a, and putting together different technological "bricks" to form a unified scheme, we believe that it should be possible to achieve, over the long term, a transfer of this type of technology to less developed countries. This 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 basic data have to be gathered, being the most usual data necessary for the design and construction of micro, mini and small hydropower schemes. They include::

- ◊ Topographic data
- ◊ Geologic and geotechnical data
- ◊ Hydrological data

In particular hydrological data have to be carefully evaluated in order to determine the two main design parameters i.e.: the optimum water discharge (Q_{design}) and the maximum water discharge (Q_{max}.), under which the plants will have to operate during their lifetime.

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

With the above mentioned information, a first rough design can be evolved. For this, the above mentioned "Lego System" will be subdivided into the following independent parts (or "bricks"):

- ◊ Water intake
- ◊ Desilting works (if required)
- ◊ Water ways
- ◊ Forebay if required
- ◊ Penstock
- ◊ Surge chamber if required
- ◊ Power house
- ◊ Tail race.

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

The first "Lego brick" **"The Water Intake"** will deal with two different types of water intakes, the first one covering intakes for the runoff-the-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 for one discharge for the tyroler type.

Above details should enable the design engineers to carry out 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. 3		SEE EXAMPLE No. 4
DROP OR TYROLER		SEE EXAMPLE No. 2					

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

Incidentally, more sophisticated desilting systems would have the disadvantage that many of their components would have to be imported. The extent of local manufacture would be considerably reduced. The cost of the structures would increase, and higher foreign exchange expenditure would be unavoidable. We have listed these simple standardized desilting structures in the following table:

TABLE 2.2
MATRIX OF EXAMPLES OF DESILTING STRUCTURES

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 TYROLER	SEE EXAMPLE No. 4						

The third "Lego brick" is the the **"Adduction Way or Canal"**. It is pertinent to recall here that a practically infinite number of combinations can be applied to the determination of canal design parameters. However, some examples given here will help the designer of specific plants to design their canals correctly and to take the necessary decisions for their execution.

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 for selecting if a tunnel is based mainly on economic factors.

Should the user of this manual required more specific knowledge in regard to canals, we recommend to consult the excellent 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 if need be, includes three examples of Design Discharges namely of 1.0, 2.5 and 3.5 m3/s

TABLE 2.3
MATRIX OF ADDUCTION WAYS AND CANAL 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 brick" 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 EXAMF No. C			

The fifth "Lego brick" is the "**Penstock Structure**", which is rather difficult to standardize, because no two penstocks are alike; The following Chapter 10 attempts nevertheless to supply useful data, in regard to both types of penstock, above ground and buried.

The following Table, which could also be completed if need be at a later date indicates an example for heads of 100-200 m with a design discharge of 1.0 and 2.5 m3/s, and examples for heads of more than 200 m with discharges of 0.90 m^3/s and 3.00 m^3/s.

Example 9 covers a penstock of the welded buried type, and gives details for its installation.

Example 10 covers a pressure test and details on the anchor blocks.

Example 11 covers a penstock with a surge tank and a relatively small discharge of 0.9 m^3/s.

Example 12 covers a penstock with a surge chamber for a discharge of 3.0 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. 7/9/10			SEE EXAMPLE No. 8/9/10			
100 - 200							
> 200	SEE EXAMPLE No. 11				SEE EXAMPLE No. 12		

The sixth "Lego brick" is the "**Powerhouse**" for which two aspects have to be considered. The first concerns the determination of the type and the sizes of the machines to be used. The 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 as for instance the "Report on standardization of small hydropower plants ", made for the UNIDO by the Swedish Consulting firm SWECO.

TABLE 2.6
MATRIX OF POWER HOUSE 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. 13			
PELTON							

The Tailrace can be handled like the third Lego brick, i.e. the waterways, Chapter 8.

Now that the proposed philosophy at the basis of this report of this report is evident, the following chapters will concentrate on the basic design criteria employed for small schemes. Some worked out examples serving as a guide, will be found at the end part of this report .

Important items like the preparation of concrete, the calculation and disposition of reinforced steel, etc., have not being covered in this report.

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 essencially 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 i.e.:

◊ 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 abovementioned 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).

Survey markers should be established with the density required to provide an adequate tacheometric survey, ensuring that continuous protection of these points is guaranteed. Benchmarks should be brass rods set in concrete foundations, protruding about 4 mm from the 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 should be carried out using conventional or electronic distance measuring equipment and theodolite.

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

Applying a tachymetric method, the density of terrain measurement 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 selection of contour lines for different terrain slopes (detail design).

TABLE 3.1
ACCURACY OF CONTOUR LINES

AREAS WITH SLOPE	USEFUL CONTOUR LINE INTERVALS
<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 map is accurate to within 0.5 mm of 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 standards.

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, permanent fix points, 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, shoreline marks towns and/or cities
- ◊ Legend of map and scale bar
- ◊ Contour interval
- ◊ Projection system used
- ◊ Geodetic datum used
- ◊ [illegible]ernment 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 establishment 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

4.1 Scope

The set of data on geology and geotechnic is an essential input to the studies. The necessary investigations normally proceed in stages as follows:

◊ desk study

◊ reconnaissance together with designers, to identify geological and geotechnical problems, to examine the general suitability of the project, to identify possible alternative sites, to define site investigations and their conditions of realization

◊ site investigations

◊ follow up during construction

The nature and amount of the necessary geological and geotechnical data should be guided by technical, not by financial considerations. The costs of investigations are low compared to the overall project costs. Moreover, well conducted investigations reduces the risks and therefore reduces costs of errors in designing and choosing construction methods.

Details on the different stages as defined above are given in the next chapters.

4.2 Desk Study

The desk study comprises the collection and the compilation of data / documents on:

Land survey:

◊ topographical maps

◊ aerial views

◊ position of survey stations, bench marks

	◊ roads, bridges (access)
Geology:	◊ geological maps
	◊ geological publications, reports
	◊ data from authorities (mines, hydrogeology, etc)
	◊ data from investigations of adjacent sites
Construction materials:	◊ existing quarries, material treatment plants
Seismicity:	◊ historical records
Geological hazards:	◊ historical records, observations from local authorities (landslides, mudflows, etc.)

4.3 Reconnaissance

A site reconnaissance carried out by the whole project team is always an excellent preliminary.

The objects of the reconnaissance are as follows:

◊ identify the sites and their possible alternatives as well as the related geological / morphological conditions.

◊ control accuracy of maps and data on topography and geology.

◊ observe and record outcrops (rock and soils), main morphological features, signs of instability of slopes, etc.

◊ record field data for further site investigations (access, water supply, sites for drill holes, etc.).

4.4 Site investigations

The content of the site investigations is always specific. It is related to the prevailing geological conditions, to the structures to be built, and to the degree of the risks that may be induced by the project. The different methods of investigations will be selected accordingly. They may include subsurface investigations such as drill holes, test pits (shafts) and even adits.

Nevertheless, a detailed geological / geotechnical **mapping** should be carried out in all cases: it is cheap and it is in most cases a very valuable source of informations.

As a guideline, following rules should be followed when planning the site investigations:

◊ The position and nature of bedrock should be known at all sites.

◊ When structures cannot be founded on sound rock, the soil profile shall be described by visual observation (test pits, drill cores) and using laboratory testing (identification, mechanical testing).

◊ The location and variation of natural groundwater level must be known at the different sites, especially, where excavations are needed.

◊ Surface power conduits need an investigation of slope stability. Generally, a detailed mapping of slope with thorough observation of all signs of instability, condition of drainage will be sufficient.

◊ Construction material i.e. concrete aggregates and embankment material are to be prospected in the inmediate vicinity of the sites. The suitability of excavation material must be studied. For concrete aggregates, a visual determination of concrete aggregate constituents will be sufficient. Durability tests and/or alkali reactivity tests will normally be made only if requested by the type of rocks encountered.

4.5 Follow up during construction

Slope cuts and excavations should be inspected by a geologist or a geotechnician during construction. It is of importance, even for small structures, to keep accurate records on their conditions of foundation and on special observations related to geology / geotechnic made during construction.

CHAPTER 5

5. HYDROLOGY AND POWER ANALYSIS

5.1 Scope

This chapter describes the required 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 power generation parameters.

The first part provides a general description of the mechanisms involved in hydrology and of the mathematical tools available to quantify the analyzed phenomena. The second part deals in some details with the flow duration curves and the available methods to establish them in practical situations. Part three explains and defines the main parameters involved in the production of energy. Finally, part four presents procedures to compute the variables to be considered whilst 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 concerned with the origin, distribution and properties of the earth's waters. 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 for the study of hydrology. At the beginning of this cycle, water is evaporated from the oceans and the land. As water vapor, 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.

Precipitation may be intercepted and evaporated by plants or run over the ground surface. This latter part either reaches a stream or infiltrates into the soil. The infiltrated water percolates to deeper zones to be stored as groundwater which later flows out from springs or seeps into streams. The streams themselves reach the oceans or enclosed water bodies, where water eventually evaporates into the atmosphere, thus completing the cycle.

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 stream. On the other hand, runoff is that part of the precipitation, as well as any other flow contribution, which appears in a given surface stream. Hence, runoff is the total flow collected from a drainage basin as it appears at the outlet of such basin.

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

The simplest input-output analysis is the determination of the water balance assuming 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:

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

The above 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 millimeters to represent the equivalent average depth of water over the basin which forms the runoff. The difference between the two represents the evaporation.

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 (5.1) has to be modified in the following way:

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

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 so called "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 defined by the following relation:

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

It represents the proportion 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 the climatic 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 themselves depend heavily on the prevailing macro-climate and are practically independent of the basin, although some variations in precipitation related to the local topography may occur and evaporation rates are influenced by the type of 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 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.

For very short periods like days or hours, the effect of basin storage becomes more 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 above description illustrates the complexity of the runoff phenomenon; adequate tools to analyze it are therefore required.

5.2.4 Characterization of the variability of runoff

The great importance of water to human existence, together with the natural variations of available water have induced man to measure and record river discharges at an early date.

This is usually achieved 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 taken on a given day. The computed discharges are then averaged to obtain the particular average daily flow. This figure is tabulated for each day of the year and, together with some additional information, is included in a series published by the agency responsible for this work. A typical presentation of such data is given in Table 5.1.

A graph indicating 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. Discharge is generally shown in m^3/s. Figure 5.1 represents a river hydrograph for the year 1980.

The hydrograph can be considered as an integral expression of the climatic and physiographic characteristics which 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 equaled or exceeded. A plot of the magnitude of discharge on the corresponding time scale (days or percent) yields the so called 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 observation period used to establish the duration curve should preferably be one complete hydrologic year or a multiple of it. It should be borne 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. The shape of the duration curve can be quantified in a variety of ways and such shape parameters can be used as analytical tools. The curve may either reflect the variability of the prevailing climate or, for basins with the same climatic characteristics, indicate the effect of the basin variables or the storage capacity.

Statistics are a very useful tool 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 provide interesting indications on the variability of runoff. A further step is then to fit 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 equaled 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 of observation: 1980

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	DEC m3/s
	AVERAGE DAILY FLOWS											
1	0.20	0.16	0.16	0.16	0.28	0.71	1.30	2.83	1.35	0.76	0.43	0.22
2	0.20	0.16	0.16	0.16	0.29	0.87	1.38	2.86	1.17	0.76	0.40	0.22
3	0.19	0.16	0.16	0.16	0.29	0.86	1.30	3.06	1.15	0.77	0.39	0.22
4	0.19	0.16	0.15	0.16	0.30	0.85	1.24	3.30	1.18	0.72	0.37	0.21
5	0.19	0.17	0.16	0.17	0.31	1.01	1.17	3.47	1.27	0.74	0.36	0.21
6	0.19	0.16	0.16	0.17	0.32	1.19	1.17	3.25	1.24	0.67	0.38	0.21
7	0.18	0.16	0.16	0.17	0.34	1.38	1.24	3.64	1.13	0.62	0.34	0.20
8	0.18	0.17	0.16	0.16	0.36	1.43	2.00	3.20	1.19	0.57	0.33	0.20
9	0.18	0.18	0.14	0.16	0.37	1.53	2.46	2.77	1.14	0.48	0.32	0.20
10	0.18	0.17	0.14	0.17	0.35	1.63	2.05	2.19	0.98	0.46	0.31	0.20
11	0.18	0.16	0.14	0.17	0.38	1.71	1.77	2.01	1.05	0.44	0.30	0.19
12	0.18	0.15	0.14	0.17	0.40	1.90	1.53	2.00	1.06	0.43	0.29	0.19
13	0.17	0.15	0.14	0.17	0.56	2.29	1.40	1.91	1.10	0.41	0.27	0.19
14	0.17	0.15	0.14	0.17	0.63	2.65	1.44	2.02	0.97	0.40	0.27	0.18
15	0.17	0.15	0.13	0.17	0.64	2.98	1.75	2.30	0.97	0.40	0.31	0.18
16	0.17	0.15	0.13	0.17	0.58	3.16	2.21	2.31	1.05	0.46	0.28	0.18
17	0.17	0.15	0.13	0.17	0.51	2.80	2.07	2.02	1.03	1.34	0.27	0.18
18	0.17	0.15	0.13	0.18	0.46	2.27	1.98	2.00	1.05	0.75	0.28	0.18
19	0.17	0.15	0.15	0.18	0.43	2.25	2.10	2.06	1.11	0.65	0.25	0.17
20	0.16	0.15	0.15	0.19	0.42	2.22	2.22	2.17	1.04	0.58	0.28	0.17
21	0.17	0.15	0.15	0.19	0.46	2.23	2.52	2.16	1.00	0.54	0.24	0.17
22	0.17	0.15	0.15	0.20	0.53	2.27	1.91	1.98	1.10	0.53	0.24	0.17
23	0.16	0.14	0.15	0.21	0.60	2.16	1.92	1.72	1.30	0.49	0.23	0.17
24	0.16	0.15	0.15	0.22	0.61	2.00	2.08	1.52	1.06	0.47	0.23	0.17
25	0.15	0.16	0.15	0.22	0.62	1.81	2.22	1.44	0.98	0.42	0.23	0.17
26	0.16	0.16	0.15	0.24	0.71	1.57	2.48	1.33	0.96	0.42	0.23	0.17
27	0.15	0.17	0.16	0.24	0.74	1.43	2.74	1.38	0.94	0.43	0.23	0.17
28	0.15	0.16	0.18	0.25	0.77	1.32	2.88	1.48	0.91	0.45	0.23	0.16
29	0.16	0.17	0.17	0.26	0.80	1.36	2.74	1.62	0.90	0.46	0.23	0.16
30	0.15		0.17	0.27	0.78	1.37	2.77	1.56	0.81	0.46	0.23	0.16
31	0.15		0.17		0.74		2.84			0.44		0.16
AVERAGE FLOW:	0.17	0.16	0.15	0.19	0.51	1.78	1.98	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	1.36	1.34	0.43	0.22
MIN. FLOW:	0.15	0.14	0.13	0.16	0.28	0.86	1.17	1.33	0.81	0.40	0.23	0.16
ANNUAL AVER.:		0.78										
PERIOD 1963-1980:												
AVERAGE FLOW:	0.13	0.12	0.13	0.25	0.81	1.66	1.88	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	0.88	0.71	0.40	0.24	0.14	0.11
ANNUAL AVER.:		0.73										

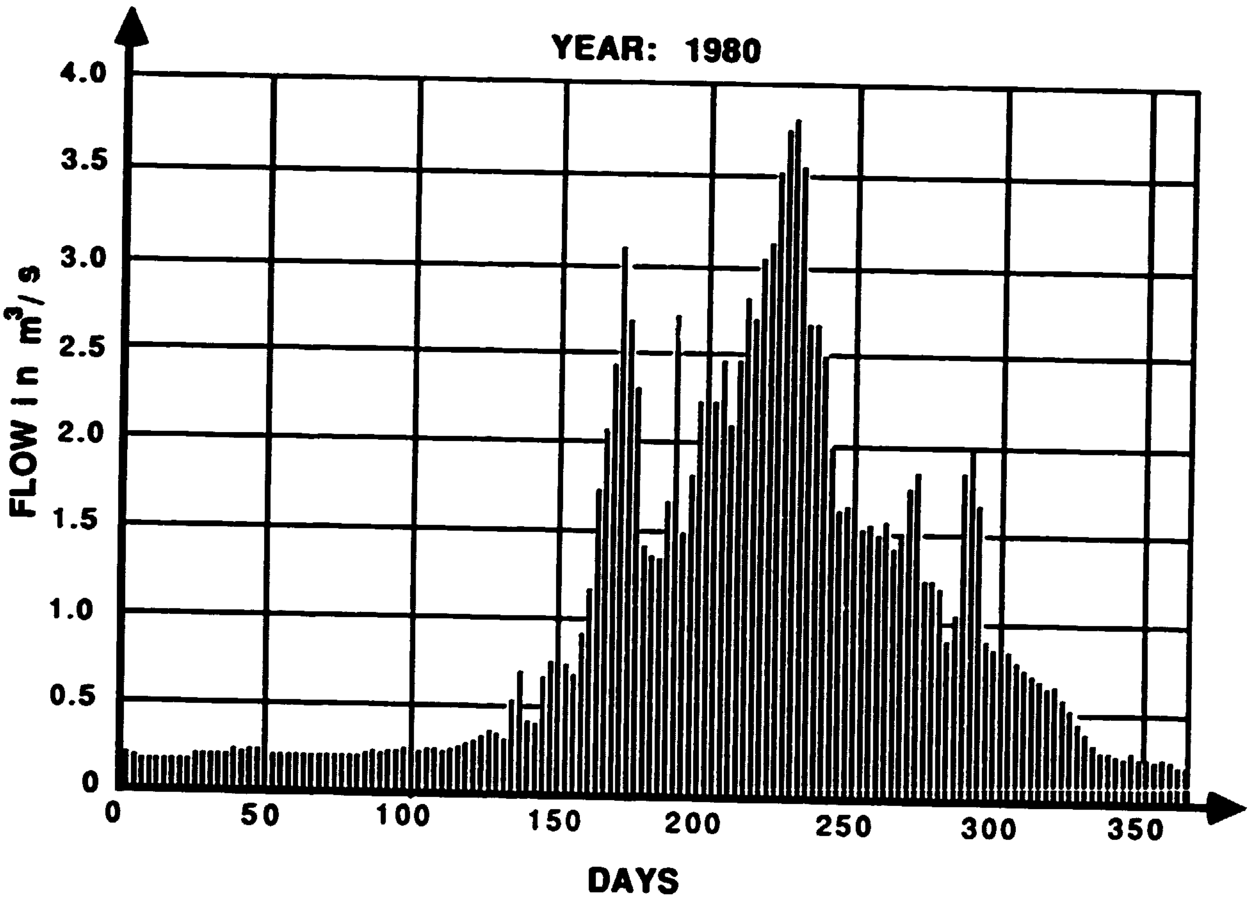


Fig. 5.1 Hydrograph

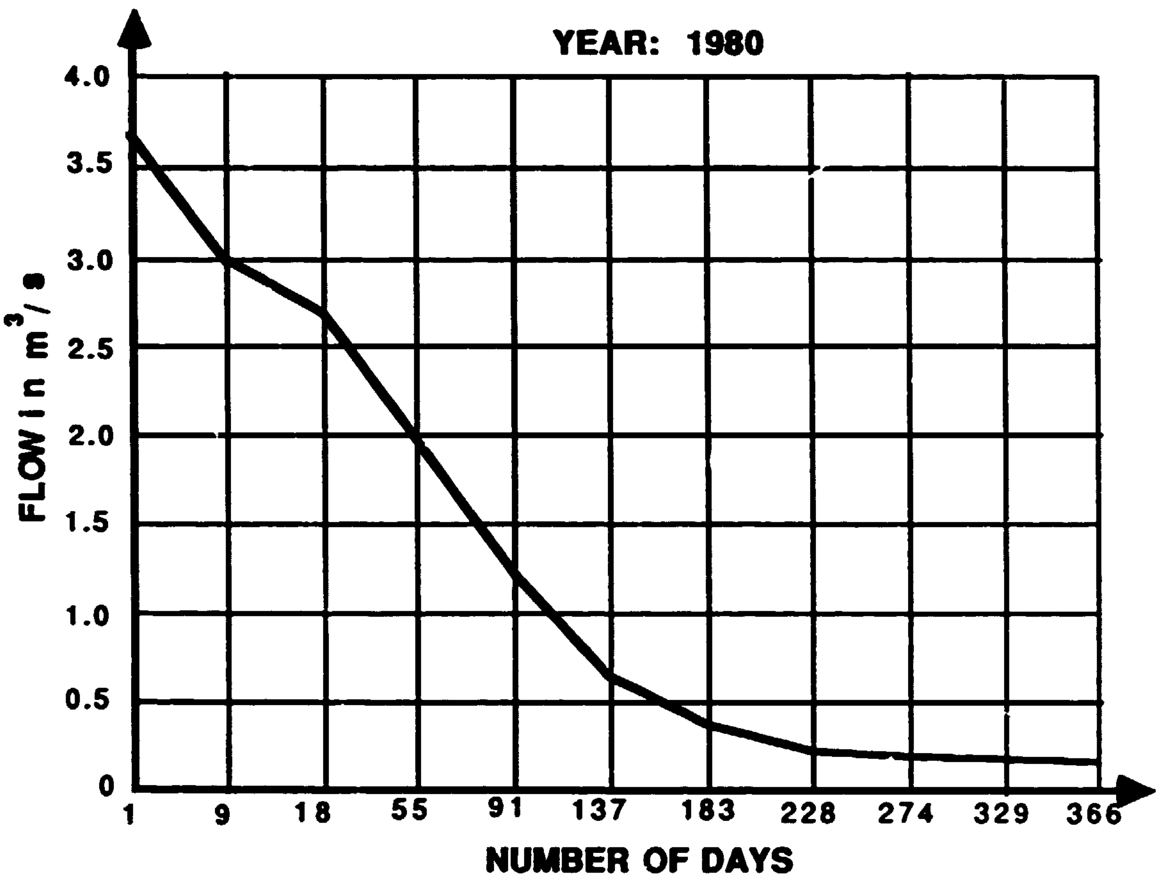


Fig. 5.2 Flow Duration Curve

5.2.5 Floods

In a period of heavy precipitation, rivers naturally experience high discharges. Often the river channel cannot accommodate 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 yet 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 instantaneous discharge, volume and duration are the main characteristics of a flood.

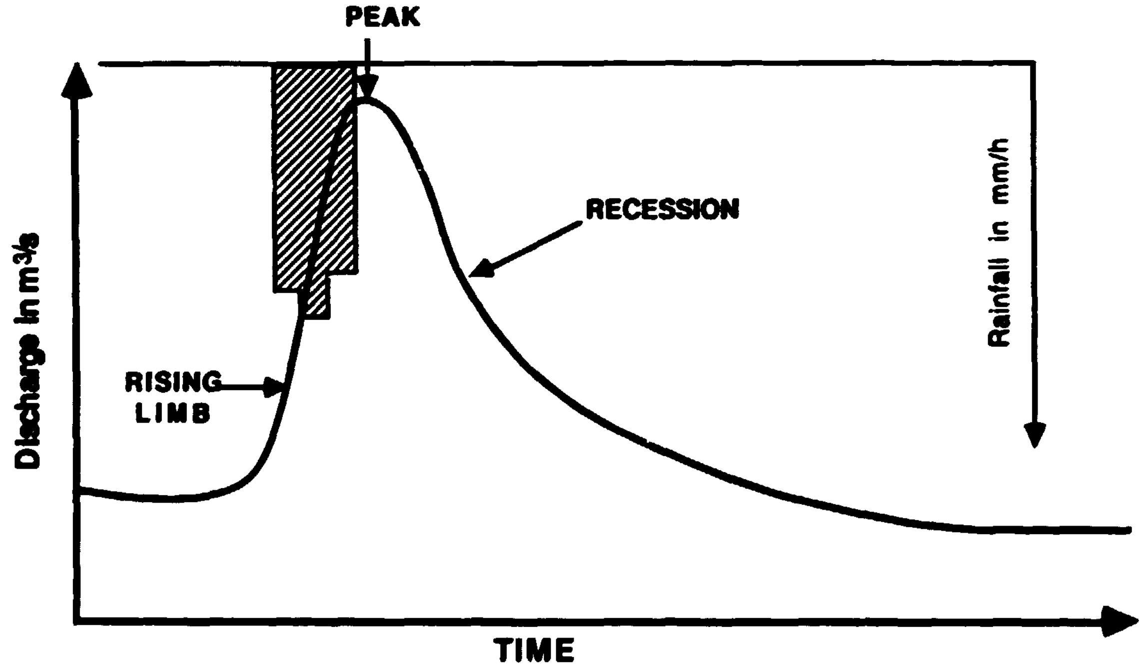


Fig. 5.3 Components of a Flood Hydrograph

A variety of factors, many of them interrelated, control the shape and dimensions of the flood hydrograph. These factors can be broadly divided into two main categories. The first category includes 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 when 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), formulae 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 in order to:

- ◊ select the installed capacity
- ◊ compute the energy production and the related dependable capacity
- ◊ perform the economic analysis
- ◊ prepare a safe design.

The parameters related to the energy production are best determined on the basis of the flow duration curve. Accordingly, the emphasis of the hydrologic studies lies on the computation of this 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, the quality of the available data should be checked.

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 confirmed 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 such verifications reveal any errors, trends and/or gaps, the discharge data must be corrected, adjusted and/or completed. Only when all these operations have been completed satisfactorily, can computation be started and the flow duration curve as indicated in Fig. 5.2, 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 to 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 also define its upper and lower limits. Thereafter, the distribution of the average annual runoff over the twelve months of the year is estimated. Then, the duration and severity of the period of low flows is evaluated. Finally the required flow duration curve is put together on the basis of the obtained results from the previous steps.

The type of data and procedures ussd in the compilation of the flow duration curve depend to a great deal on the specific 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 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 may be either runoff and precipitation in the project basin, and/or runoff measured both in the project and in a 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, however, 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 Formulae

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, the basic formulae and definitions are presented hereafter.

Energy is created as a result of water falling from a higher to a lower level. Power is the rate of energy generation. The following equation may be written :

$$P = 9.81 \cdot Q \cdot h \cdot e \tag{5.4}$$

where:

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

***Note:** 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 larger 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 of water to the power plant falls. However, a distinction has to be made between the following related terms:

The **headwater elevation** means the elevation of the water surface in the forebay structure (intake), from which the water is released 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** results from 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 operational 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 amounts to about 30 to 50 % of the design discharge (see Fig. 5.4).

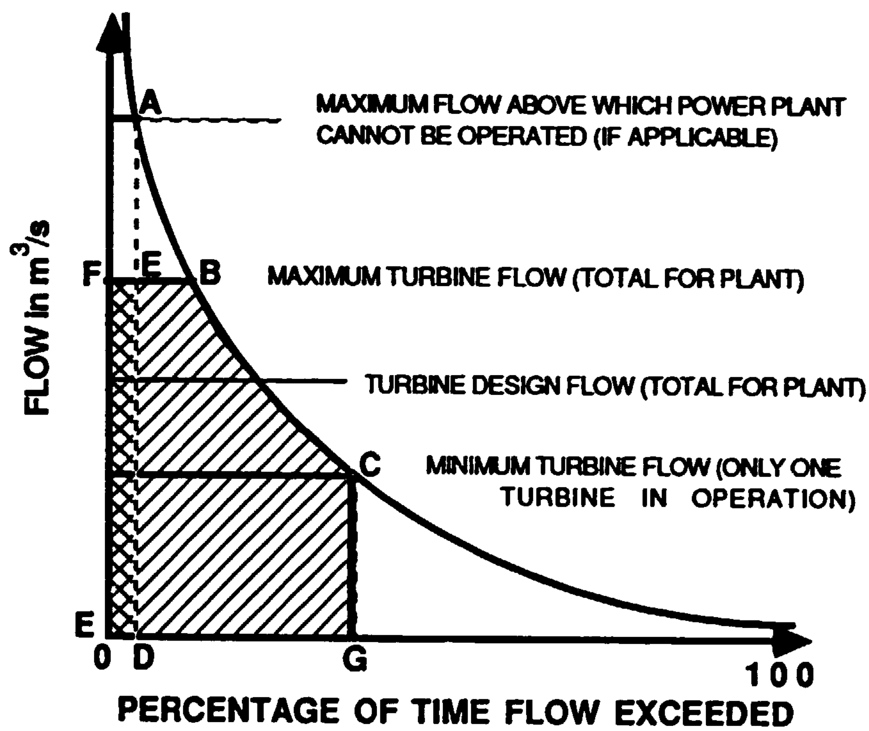


Fig. 5.4 Flow Duration Curve and Energy Production

The **rated capacity** is defined by 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 m^3/s
h = rated head, in m
e = efficiency of the powerplant.

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 whether the incremental energy production and whether the incremental capacity can effectively be absorbed by the distribution system (not all the power plants can operate together in the peak period). 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 certain rules of thumb. According to these, the optimal plant capacity respectively design discharge generally lies within a range of discharges defined on the flow duration curve by the 35% and 25% excess 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 Energy Production

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 5.4 The procedure is as follows :

◊ Define the points **A**, **B**, and **C** on the flow duration curve, corresponding 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** at the intersections 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 :

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

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

Once the average annual energy production is known, the plant factor can be computed, using the definition given in paragraph 5.4.3 above.

5.5.3 Dependable Capacity

The value of the delivered power depends upon whether 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, a few general concepts are presented below, mainly to alert the readers to the problem.

Basically, three elements play a role in this respect: 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 also 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, variations of these parameters from year to year, statistical analysis). The results of such computations will show how critical this issue really is and whether it is worth-while 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). At the design stage however a very careful study is necessary in order to avoid future problems An incorrectly designed water intake may affect the whole plant to such an extent as to make it useless.

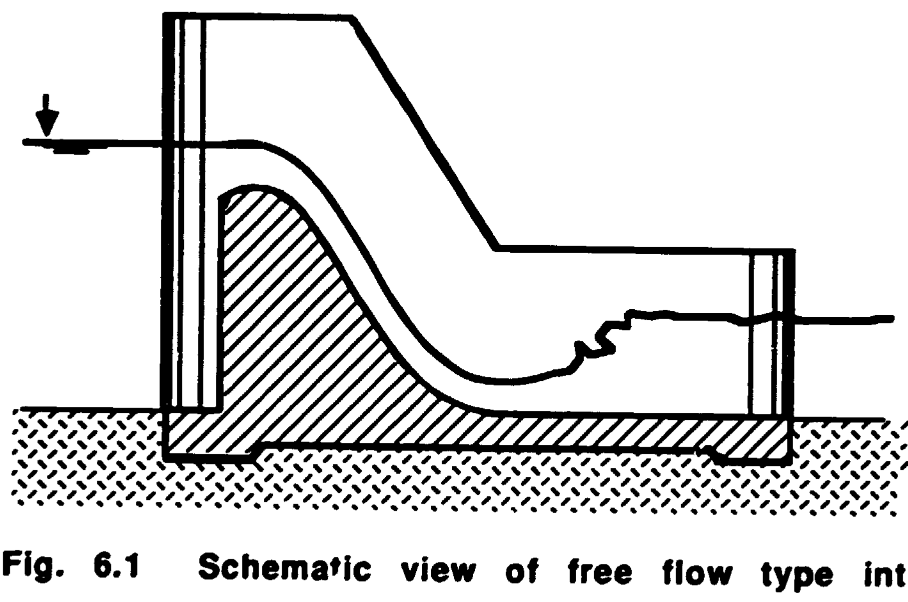


Fig. 6.1 Schematic view of free flow type intake

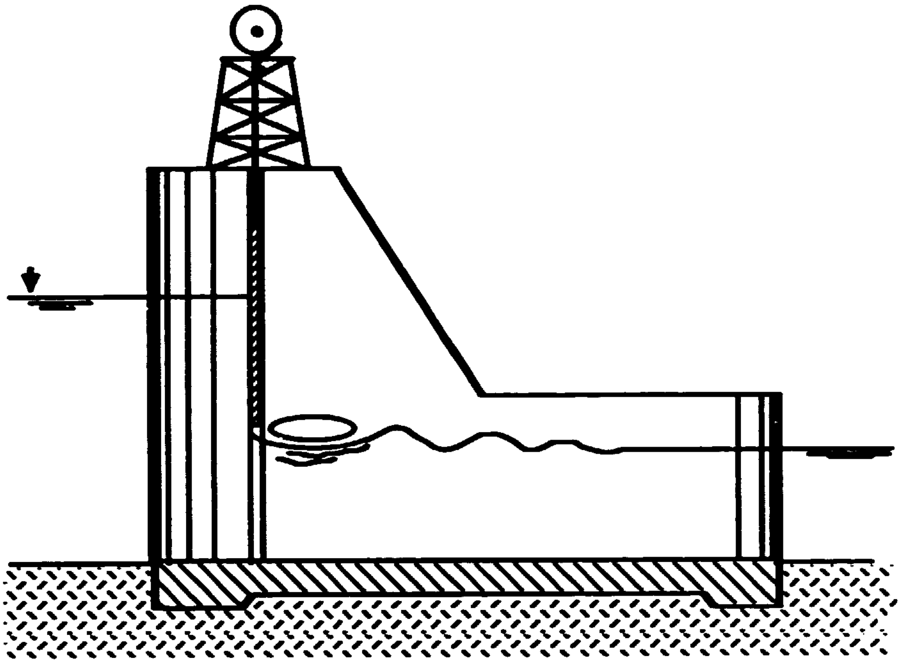


Fig. 6.2 Schematic view of submerged flow type intake

For locating of the intake structure, the design engineer should consider the following factors:

- ◊ Topographic conditions
- ◊ Geological and geotechnical conditions including the nature of the stream bed
- ◊ Natural bends along the river
- ◊ Water rights and water used for other purposes
- ◊ Accessibility 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 will be most suitable. The possibility of slope instabilities in the immediate vicinity of the structures should be investigated, in order to avoid such areas , or to protect them if they cannot be avoided.

6.1.2 Geological and geotechnical conditions

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

6.1.3 Nature of the stream bed

The elevation of the stream bed must 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 of erosion. A significant erosion of the stream bed may lower the elevation at the intake entrance to such an extent as to prevent the desired water from entering the intake (see Fig. 6.3).

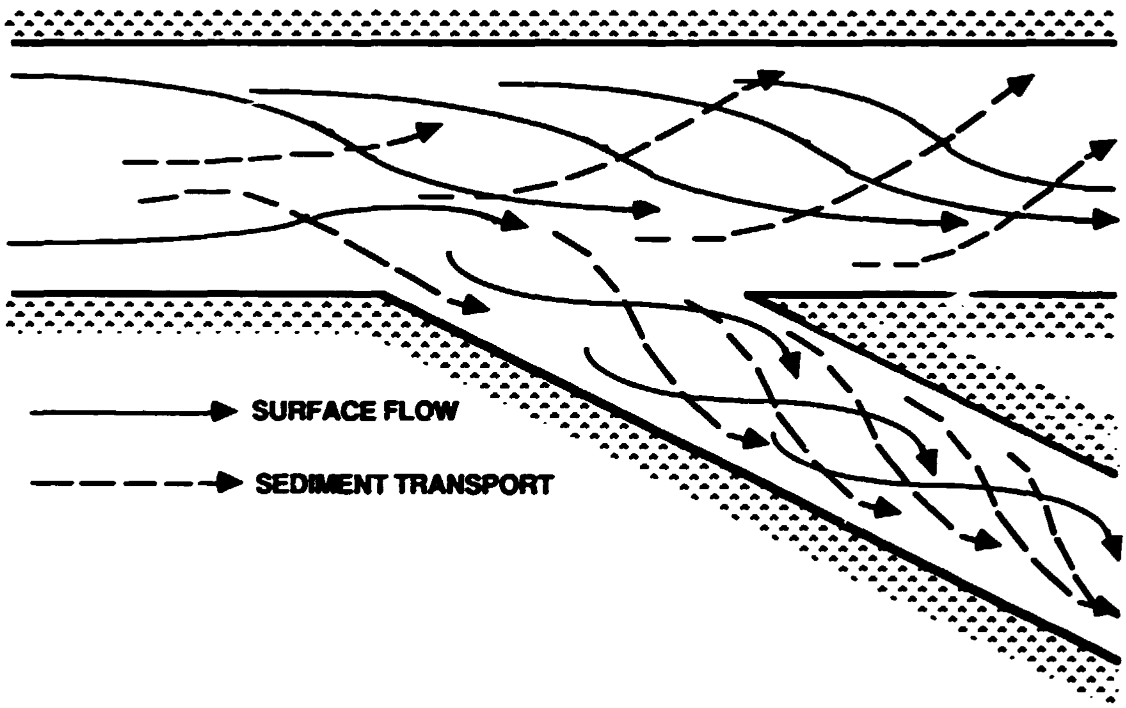


Fig. 6.3 Surface flow and sediment transport conditions of a straight river section with a lateral branch.

The above danger can be avoided by locating intake and weir in an area where the geological and geotechnical studies indicate that the stream bed will be stable and permanent. Particularly favourable conditions are obtained where the stream flows over a bed rock or where the river flow varies little throughout the year, small slope flows also simplify the intake design.

However, in most cases erosion as well as sediment transport are present, and therefore 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 the effect of a weir built on a river bed carrying sediments may cause obstruction in front or permanent damage to an intake; therefore provision has to be made to let the excess of sediments pass across the weir or barrier without entering the intake.

The selection of the site or location of the intake should ensure that not only the above technical conditions are met, but that at the same time costs should be minimized; effectively the expenditures involved depend on the structure size and volume, as well as the method of construction; a simple but effective structure should be aimed at (see Fig. 6.4).

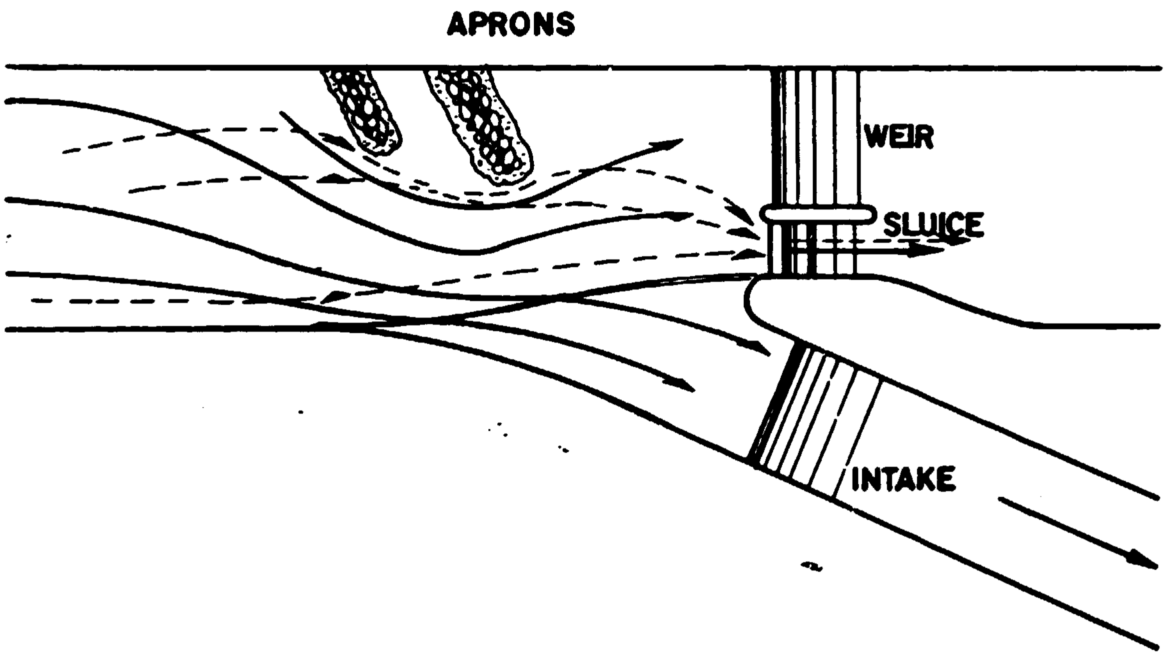


Fig. 6.4 Correction of a straight river with the help of aprons and training walls

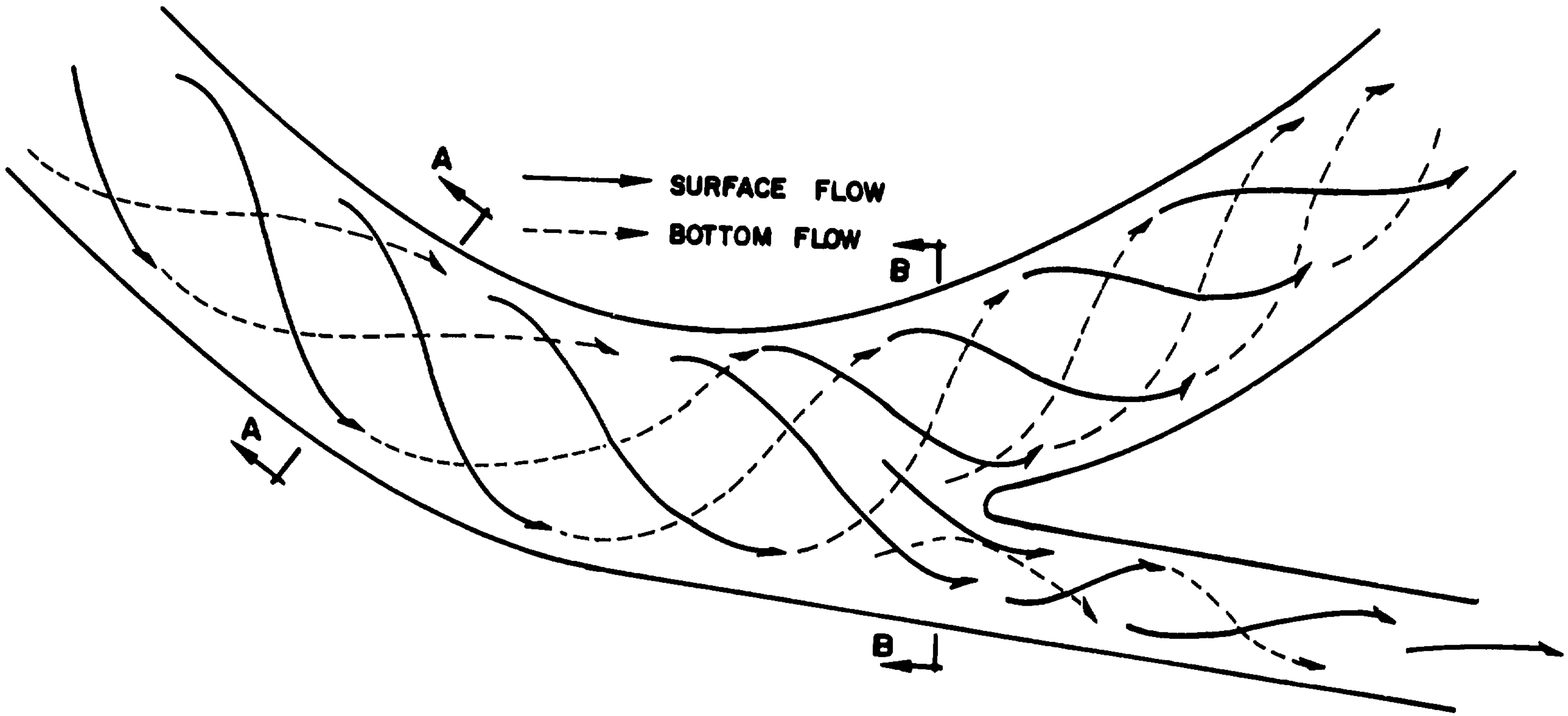


Fig. 6.5 Stream flow in a curved section of a river

6.1.4 Natural bends of the river

The orientation and location of an intake in a river will be decisive in regard to its success or failure . Advantage should be taken of the natural bends of the river, placing the water-intake whenever possible at the outside of a bend or, on a relatively straight section of a stream, with the entrance of the water intake oriented laterally (parallel to the flow) and not facing towards upstream (i.e. perpendicular to the flow).

River bed sediment transport is a main problem to be considered during the design phase of an intake structure. Because in a bend of a river the surface stream flow moves, laterally and faster than the stream flow on the bottom of the river, the water flow rotates like a corkscrew (see Figs. 6.5, 6.6, and 6.7), the sediments moving in a more or less opposite direction. Floating materials such as wood, can be easily diverted, or removed with the help of trash racks. However, granular materials and sediments have a tendency to form deposits in front of the intake entrance.

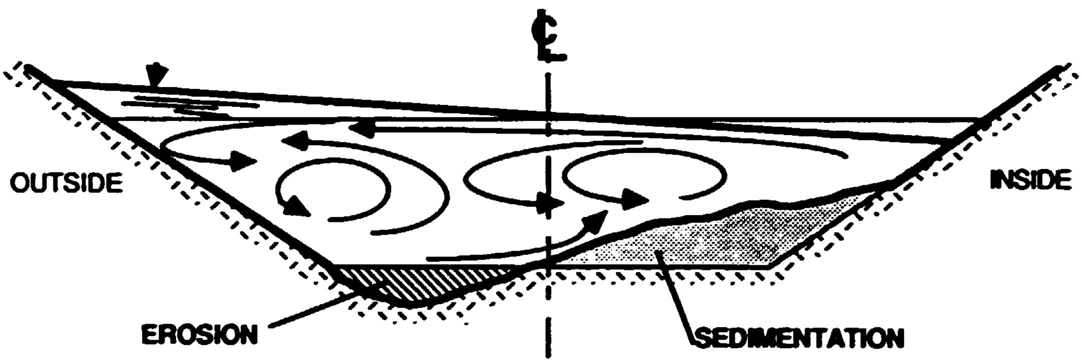


Fig 6.6 Cross flow on a section A - A

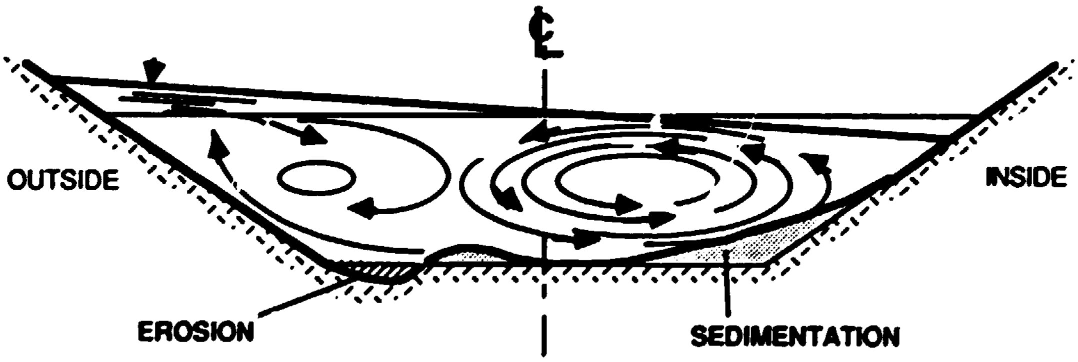


Fig. 6.7 Cross flow on a section B - B

For the above reason, it is convenient to create a device to retain the sediments first, and to channel them afterwards through a passage specially designed and constructed, thus, allowing to maintain the front of the water intake clear of sediments. Depending upon size and importance of the intake structure, the evacuation of sediments - *which should be carried out with as little water losses as possible* - will be done manually or machanically, (See Fig. 6.8).

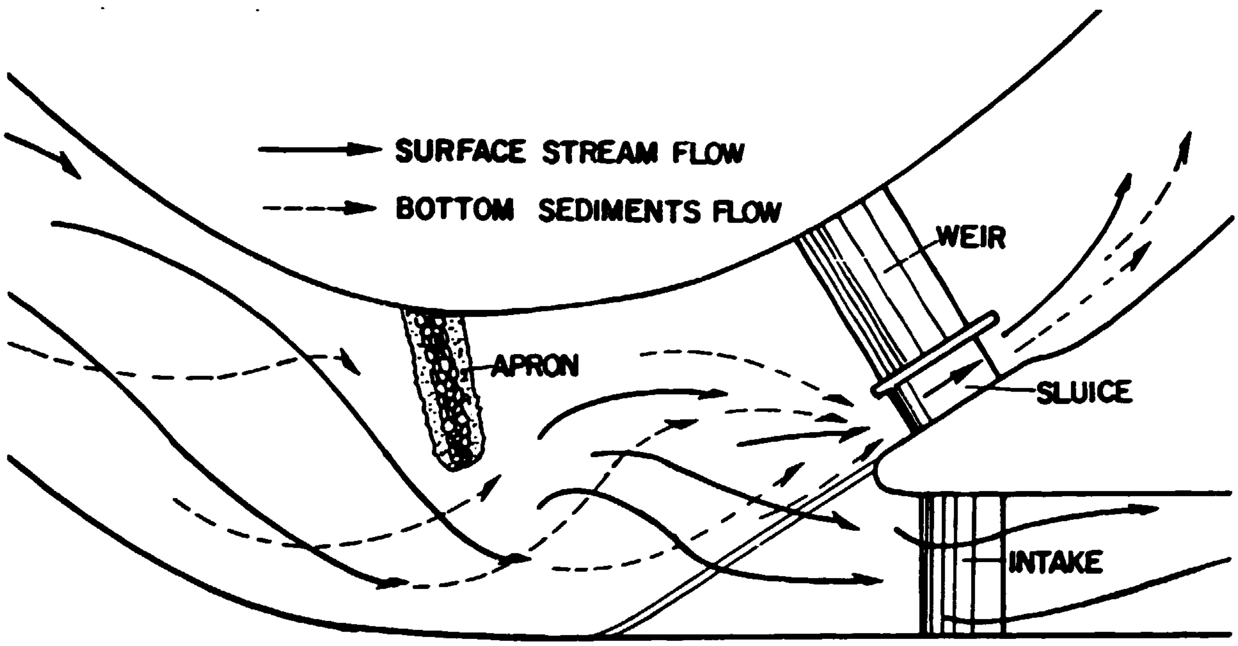


Fig. 6.8 Melioration of flow conditions at the intake in a river bend by means of an apron

From experience facts have been deducted which should be observed when designing a water intake; they are as follows:

1. If possible, it is advisable to place the water intake on the outside elbow of a stream.
2. As the water intake slows down the transport of the sediments, the sharper the curve of the stream flow, the greater will be the retardation of the sediment transport.
3. Usually the most efficient location of the water intake is either the lower part of a river bend, or in a straight section, immediately after the apron or training wall, but on the opposite site.
4. The design and lay-out of a water intake that takes into account the above conditions are most efficacious when flushing and clearing the intake of sediments accumulated in front of its entrance are considered.

6.1.5 Water rights and other water uses

It is important to clarify the legal implications concerning present and future water rights, before the owner starts actual construction.

If other water requirements (water supply, irrigation, etc.) are envisaged or have been granted in the past, adequate compromise solutions must be agreed to. A particularly elegant solution is to pass the water in the turbines before it is used for irrigation or general water supply. Should this not be possible, necessary provisions will have to be made and the intake will have to be complemented with bypass facilities.

Rules and regulations may 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 water users must be established.

If irrigation and power generation each require a substantial percentage of the available water it might be necessary to generate 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 ensure permanent access for operation and maintenance. 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 (1) useful water to be captured by the intake, and (2) 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 flow into the original river course. For power generation it is desirable to utilize as clear as possible water; the bottom sediments have therefore to be flushed periodically in order to keep the intake entrance free and unobstructed, an operation that causes 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).

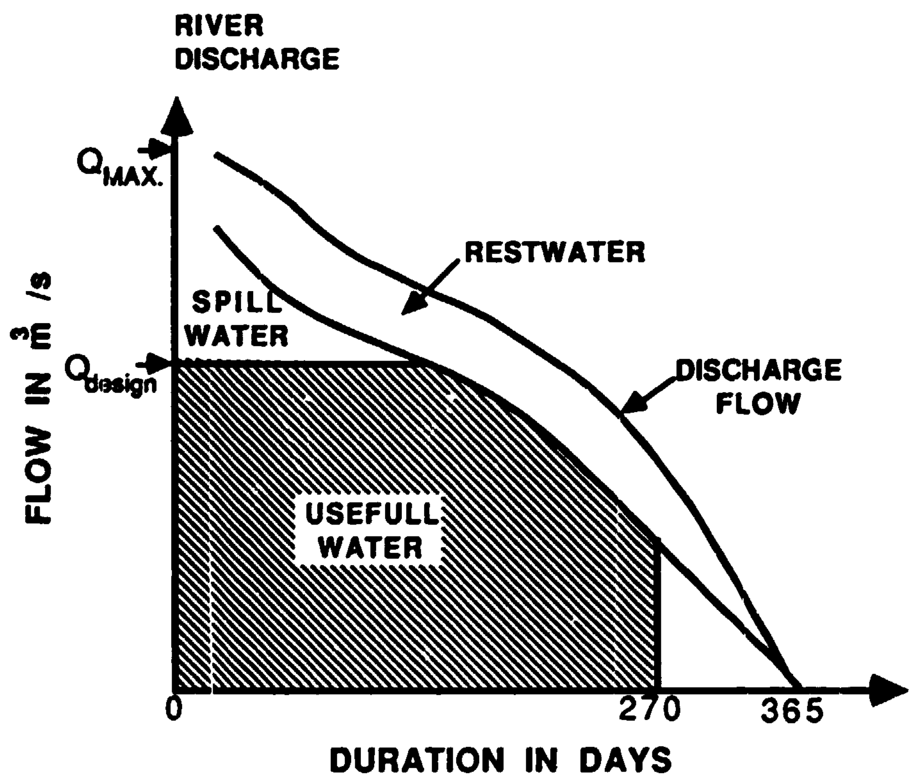


Fig. 6.9 Determination of Q_{design} with a flow-duration curve.

The Q_{design} or design discharge corresponds to the water quantity used to dimension the intake structures. It is the maxir¨ım water discharge which can be diverted from the main stream. The structure also limits the amount of water diverted from the total incoming water during the occurrence of floods.

The hydrological study has to take into consideration the water requirements for other uses and to determine their importance. An optimization of the use of water and its $Q_{design.}$ value should be gained from an economic study.

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 a 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. The outside will be eroded and deposits of sand gravel and other sediments will occur on the inside (See Figs. 6.5, 6.6, 6.7, and 6.8).

This fact, can be used for the scheme's benefit the maximum of clear water can be concentrated into the intake entrance, and sediment transport will be deviated from the intake entrance and guided directly to the flushing gates. However this principle as simple as it appears should be properly applied; by neglecting minor factors the entire purpose of the intake may be seriously jeopardized.

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, mobile or of a mixed type).
◊ Intake entrance.
◊ Forebay.
◊ Flushing channel.

6.2.3.1 The Weir

The weir or barrier is usually built 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 spill the water excedents. Because of topographic or/and geologic restrictions, it may be sometimes necessary to arrange the weir in a position that is not perpendicular to the stream flow of the river. This is acceptable provided that the economic restrictions as well as the technical requirements are properly met.

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 forces 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. An energy dissipator or rip-rap protection should be provided, in cases where erosion is expected.

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

$$Q = \frac{2}{3}\,\mu\, L\, \sqrt{2g}\left[\sqrt{H^3} - \sqrt{\frac{v^2}{2\,g}}\right] \qquad (6.1)$$

Where the approach velocity is small, this formula can be simplified as follows:

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

or

$$Q = C\, L\, \sqrt{H^3} \qquad (6.3)$$

were: μ = Discharge coefficient (dimensionless)

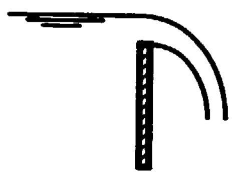
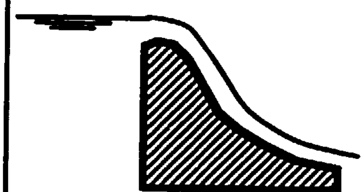
L = Width of the spillway or weir (m).

g = Acceleration of gravity (= 9.81 m/s^2).
H = Energy head above the spillway (m).

$$C = \frac{2}{3}\,\mu\,\sqrt{2\,g}\ \ (m^{0.5}/\,s) \qquad (6.4)$$

The coefficient μ, respectively the factor C, fluctuate in between the values shown in Table 6.1:

TABLE 6.1
SPILLWAY COEFFICIENTS

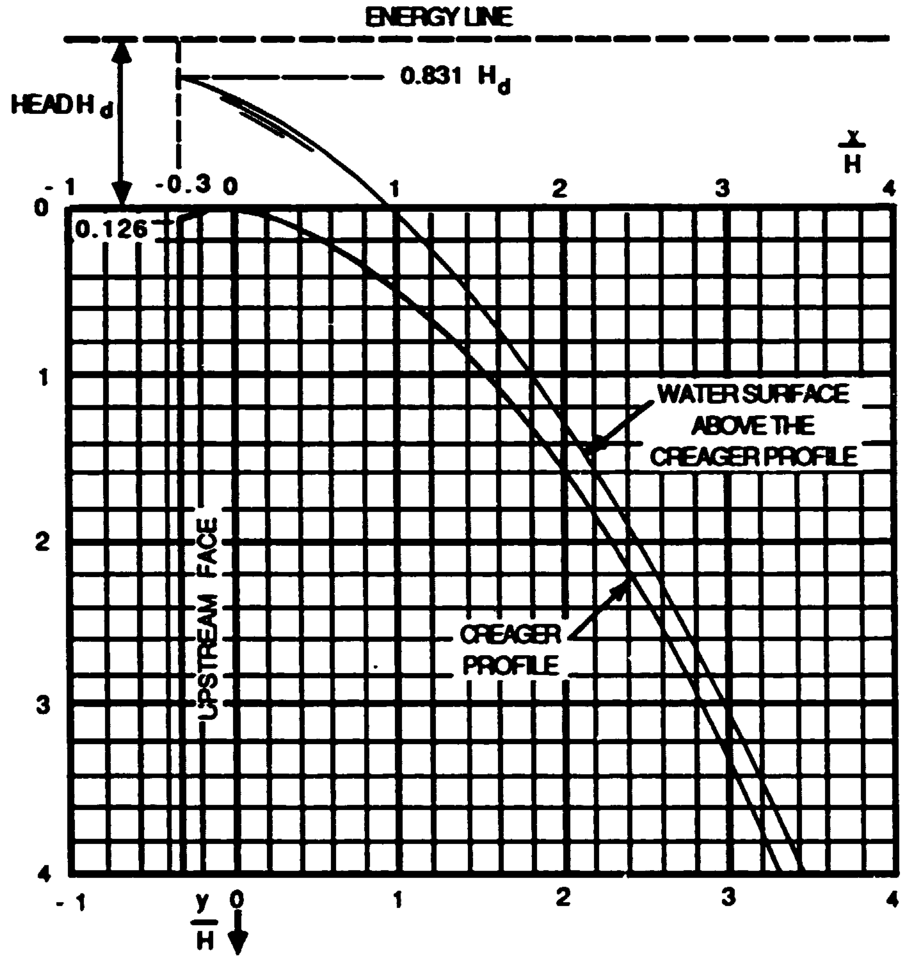
FORM	 SHARP	 O.G. or FLAT
μ	0.60	0.65 - 0.75
C	1.80	1.92 - 2.20

6.2.3.2 Design form of the Spillway or weir

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

Based on experience, a standard form for small weirs has been developed which is appliable in most cases, see Fig 6.10.

(H_d is the design energy head above the spillway).



$\frac{x}{H}$	$\frac{y}{H}$	$\frac{y_w}{H}$
-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

$\frac{x}{H}$	$\frac{y}{H}$	$\frac{y_w}{H}$
0.900	0.397	-0.220
1.100	0.565	-0.050
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, or small gravity dam (if necessary) and/or other intake structures, it is necessary to determine the forces which may be expected to affect the stability of these structures.

The main forces which must be taken into consideration are:

- ◊ External water pressure
- ◊ Internal (or uplift) water pressure
- ◊ Sediment pressure (if any)
- ◊ Ice pressure (if any)
- ◊ Self weight of the structures
- ◊ Additional pressure if any, caused by earthquakes.

With overflow in ungated weirs the total horizontal water pressure on the upstream face of the weir is given by the trapezoid shown in Fig. .6.11 The unit pressures at the top and at the bottom are γh_1 and γh_2 respectively where γ is the unit weight of water assumed as 1000 kg/m^3 (10 kN/m^3).

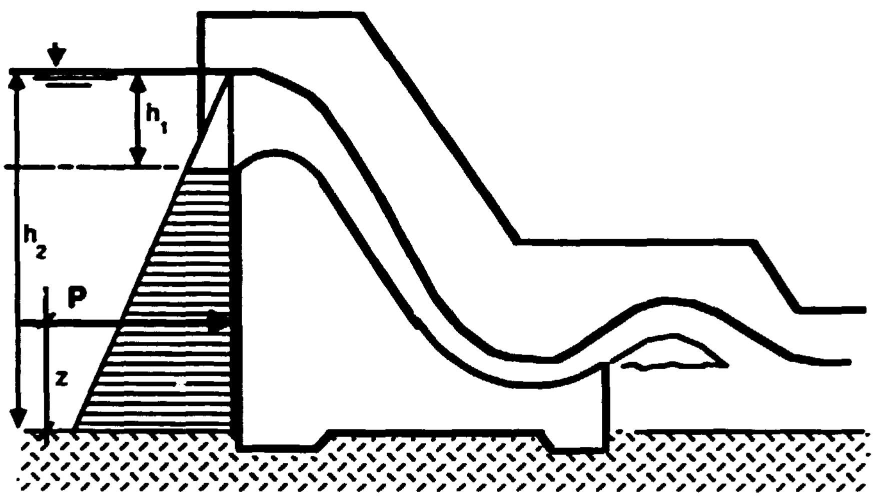


Fig. 6.11 Water pressure acting on an ungated weir

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

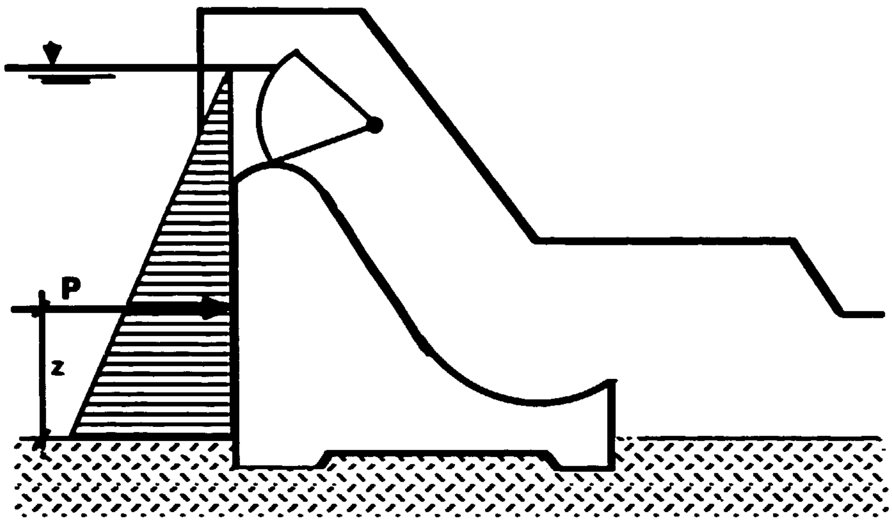


Fig. 6.12 Water pressure acting on a gated weir

6.2.3.4 Requirements for stability

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

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

Overturning of a weir or a dam occurs when the resultant force of all horizontal and vertical acting forces on the structure pass outside 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 pressure at the upstream edge of any horizontal section computed without uplift, exceeds the uplift pressure at that point. If this is the case, the weir or the structure may be considered to be safe against overturning. If the uplift pressure at the upstream face exceeds the vertical pressure in any horizontal section computed without uplift. The uplift forces on the plane of an assumed horizontal fissure will increase 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 concrete base of the structure and the foundation soil or rock.

In medium large and in large structures a criteria of the shear friction factor is used which usually requires an extensive 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 U. S. Bureau of Reclamation, recommends to check the structures against horizontal displacements by calculating the so-called allowable sliding factor method.

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:

$$f = \frac{\Sigma V}{\Sigma w - U} \qquad (6.5)$$

w are the weights and U the uplift pressure

The Bureau of Reclamation in its excellent Book "Design of Small Dams" gives the the allowable sliding factors between concrete and various foundation materials as shown in Table 6.2. These are conservative figures and may be used when laboratory tests cannot be made:

TABLE 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

Source: "Design of Small Dams" Bureau of Reclamation,.USA.

It is important to mention that he geological investigations should find out 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 constituting the weak stratum.

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

The foundation material in medium and large-sized structures must be investigated in situ by experienced soil engineers; enough samples are to be taken to enable a thorough analysis in the laboratory. In the case of small structures this type of procedure may however, not be economical and it is then recommended to use the allowable bearing pressures given in 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 used as a guide for design and quick check only and are not intended to replace the sound advice of experienced local 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 6.3

ALLOWABLE BEARING PRESSURES FOR DIFFERENT TYPES OF SOILS

MATERIAL	ALLOWABLE BEARING PRESSURE (kg/cm^2)
Sound, massive igneous methamorphic 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 foundation of the structure. These water pressures will act in all directions and can sometimes reduce the resisting forces considerably. Uplift pressures, therefore, should be included in any stability analysis of the structures.

Internal water pressures will be different if the foundation of the structures are supported on a solid rock or 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 structure is equal to the full water head; at the downstream face it equals zero if no tailwater is present; a linear distribution is assumed between the upstream and the downstream face. The correct placement of drains on the structure 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 such cases, additional works are required, such as cutoff walls, aprons, etc., in order to provide enough safety against potential seepage and piping.

Concrete gravity structures with more than 7 m height on pervious foundations require detailed and extensive soil mechanics investigations. Such cases are beyond the scope of this report and should be handled 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 only by trenching (with machines or by hand) and forming the concrete wall into the excavated trench.

It is recommended to back fill the trench surrounding the wall with an impervious well-compacted material. If the trench is hand made, provision should be made to protect the works by strutting the trench walls during excavation and construction , thus preventing any serious accidents due to sudden collapse of the trench walls.

Cutoff walls should be always constructed on the upstream side of the foundation (*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 Lane's weighted creep theory. It is an empirical method based on a statistical analysis of 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 five main 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 difference.

(3) Reverse filter drains and pipe drains are recommended aids to higher security from under seepage. They can reduce the weighted creep ratio by as much as 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 dam on its foundation is proportional to the weighted creep distance

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

TABLE 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 Fig. 6.13.

By definition:

$$\text{Weighted creep ratio} = \frac{\text{Weigthed length of path}}{\text{Head difference}}$$

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

(see Fig. 6.13)

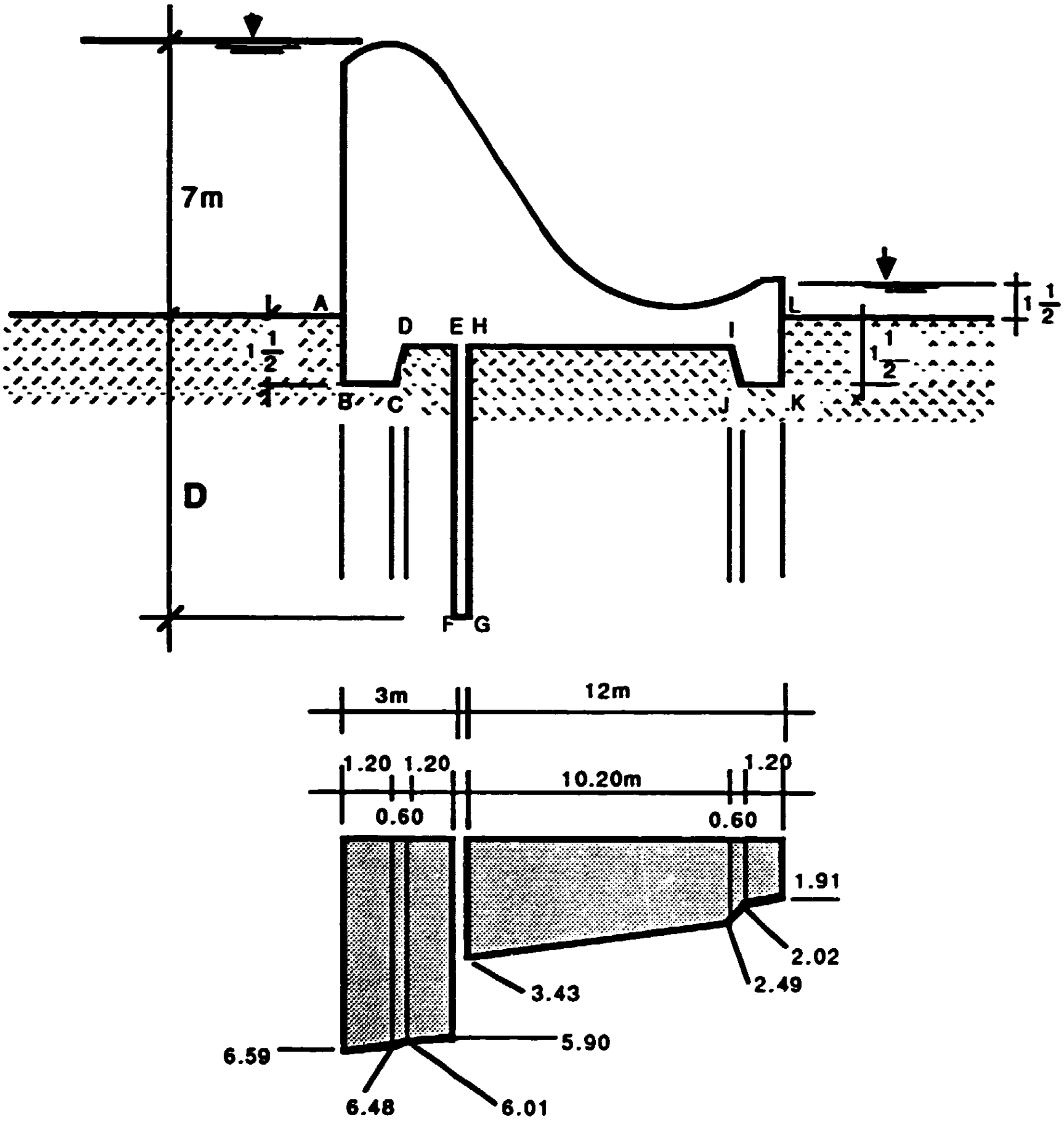


Fig. 6.13 Application of Lane's Weighted Creep Theory

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

Σ Vertical path = 4 (AB) + 2 (EF)

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**

Difference in water head = (7 - 1.5) = 5.5 m

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

or:

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

If the soil is a Medium Gravel with WCR = 3.50 (see Table 6.4)

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

If the soil is a Coarse Sand with WCR = 5.00 (see Table 6.4)

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

According to Lane's recommended ratios (see Table 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 better visualize 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 difference
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

$$\text{Uplift at point B} = 5.5 - \left[\left(\frac{1.5}{20}\right) x\, 5.5\right] + 1.5 = 6.59\ \text{m}$$

$$\text{Uplift at point C} = 5.5 - \left[\left(\frac{1.5 + \frac{1}{3} x (1.20)}{20}\right) x\, 5.5\right] + 1.5 = 6.48\ \text{m}$$

$$\text{Uplift at point D} = 5.5 - \left[\left(\frac{1.5 + \frac{1}{3} x (1.20 + 0.60)}{20}\right) x\, 5.5\right] + 1.5 = 6.01\ \text{m}$$

$$\text{Uplift at point E} = 5.5 - \left[\left(\frac{1.5 + \frac{1}{3} x (1.20 + 0.60) + 1.5 + \frac{1}{3}(1.20)}{20}\right) x\, 5.5\right] + 1.5 = 5.90\ \text{m}$$

$$\text{Uplift at point H} = 5.5 - \left[\left(\frac{1.5 + \frac{1}{3} x (1.20 + 0.60) + 1.5 + \frac{1}{3}(1.20) + 4.5 + 4.5}{20}\right) x\, 5.5\right] + 1.5 = 3.43\ \text{m}$$

$$\text{Uplift at point I} = 5.5 - \left[\left(\frac{1.5 + \frac{1}{3} x (1.20 + 0.60) + 1.5 + \frac{1}{3}(1.20) + 4.5 + 4.5 + \frac{1}{3}(10.2)}{20}\right) x\, 5.5\right] + 1.5 = 2.02\ \text{m}$$

$$\text{Uplift at point J} = 5.5 - \left[\left(\frac{1.5 + \frac{1}{3} x (1.20 + 0.60) + 1.5 + \frac{1}{3}(1.20) + 4.5 + 4.5 + \frac{1}{3}(10.2 + \frac{1}{3}\, 0.6)}{20}\right) x\, 5.5\right] + 1.5 = 2.02\ \text{m}$$

$$\text{Uplift at point K} = 5.5 - \left[\left(\frac{1.5 + \frac{1}{3} \times (1.20 + 0.60) + 1.5 + \frac{1}{3}(1.20) + 4.5 + 4.5 + \frac{1}{3}(10.2) + \frac{1}{3}(0.6) + 1.5 + \frac{1}{3}(1.2)}{20}\right) \times 5.5\right] + 1.5 = 1.91\text{m}$$

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/m3

P_T = 52 640 kg/l m

or **P_T = 52.6 Ton/l 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 Fig. 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 gorges with sound foundation rock or reliable foundation materials on which to place the intake structure offer convenient and economical solutions. It is evident that drop intakes are particularly well suited for locations in steep terrains.

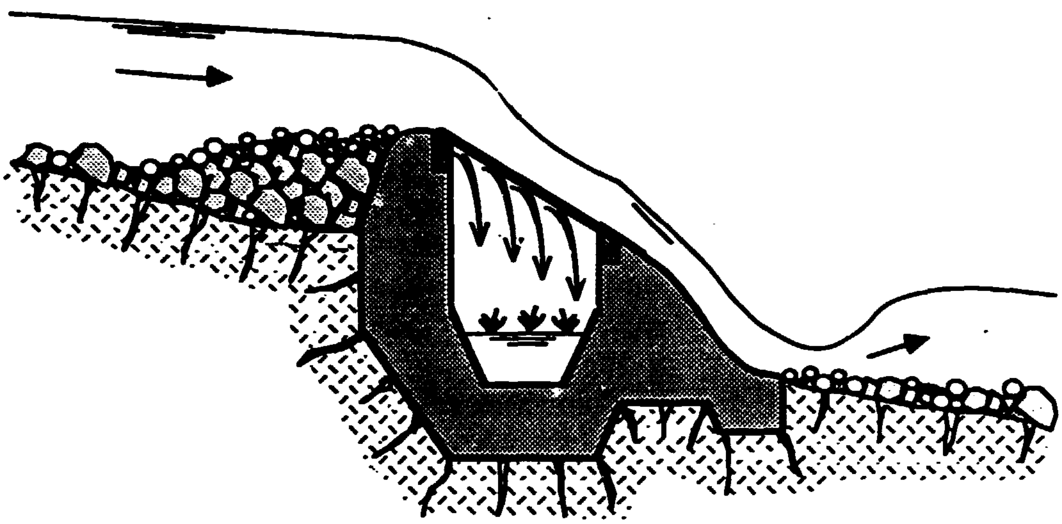


Fig. 6.14 Principle section of a drop intake

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. For most of the drop intakes a considerable larger amount of sediments must be expected than the run-off-river type intakes.

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

In this report the design criteria adopted for drop intakes, deviates very little from the standard designs made by TIWAG in Austria or EDF in France. The only difference is the slope angle of the entrance rack. For small discharges ($Q < 3$ m3/s) the writers recommend to use a slope ·ʼ 70% instead of 80% used for larger discharges. This value of 70% is based on the experience gained in other similar intakes like Otemma, Breney, Leteygeon, Stechelberg, etc.

The recommended opening between rack bars is 25 mm for discharges of Q < 3.5 m3/s (see Fig. 6.15).

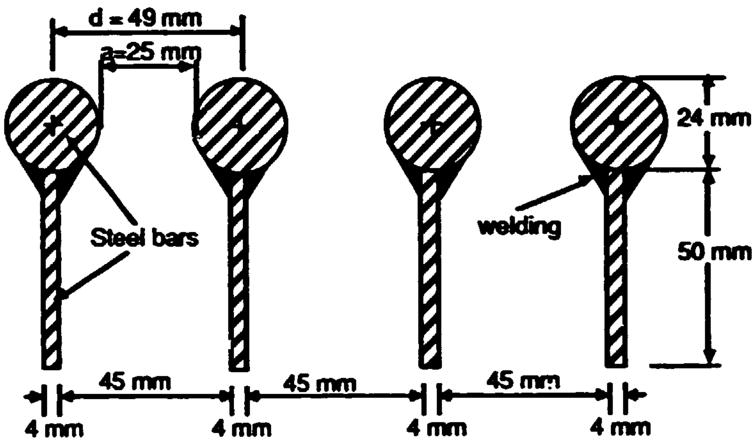


Fig 6.15 Typical rack for a drop intake with Q < 3.5 m^3/s

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 on the average 10% larger than the desilting basin with the same Q_{design} but for a run-off-river intake with a vertical side rack, because 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 generally excavated in rock to produce a more regular and turbulence free approach of the water flow on the rack.

◊ Two openings with slots which can be closed by 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.

◊ A device for inserting 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 and the rack cleane device must be checked from leaves and gravel at least once a week; further the thickness of the sand-gravel deposit should be checked at least at three points in the desilting basin. When necessary the flush gate should be opened.

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

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

If the river or creek is known for flash floods, it is recommended to :

- Place the intake on the outside of the river bend.
- Prepare a special upstream protection wall.
- Adapt the design of the intake for special occurrences such.as large rocks, tree trunks, pumice stones, etc.

6.3.2. Rack size and discharge capacity

The dimensions of the rack can be estimated as follows :

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

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

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

$$h = \frac{3}{4}\, h_c \qquad (6.7)$$

The discharge capacity is estimated with the formula:

$$Q = \frac{2}{3} C \mu \beta \ell \sqrt{2 g h} \qquad (6.8)$$

where:

h = water depth at the top of the rack = 3/4 . h_c
h_c = critical depth
c = 0.6 . (a/d) . $(\cos \beta)^{3/2}$ = 0.6 . (25/49).$(\cos 35°)^{3/2}$ = 0.23 (see Fig. 6.15)
μ = rack coefficient = 0.90
B = rack width
ℓ = design rack length
L = total rack length
β = angle of the rack with the horizontal
a = opening between rack bars (see Fig. 6.15)
b = distance between bars centers
g = gravitational acceleration constant (9.81 m/sec^2)

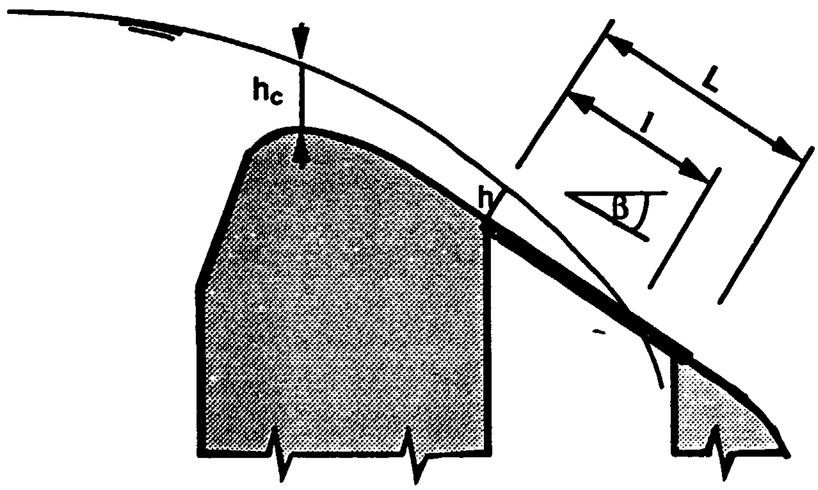


Fig 6.16 Depth and length of the rack of a drop intake

For drop intakes with a Q < 3.5 m3/s the formula for Q can be simplified:

$$Q = (0.66)\ (0.23)\ (0.90)\ \sqrt{2 g}\ B\ \ell \sqrt{h}$$

or

$$Q = 0.61\ B\ \ell \sqrt{h} \qquad (6.9)$$

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 at least 1.3 (because of the possibility of a partial obstruction of the rack, possible negligence in the maintenance, and for reserve flow during flushing operations).

Practical experience shows that :

$$L \text{ approx.} = 1.3\ \ell \tag{6.10}$$

and

$$B \text{ approx.} = 0.60\ L \tag{6.11}$$

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

6.3.3. Operation of drop intakes

A drop intake functions 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 limits 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 is 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 at the entrance to the penstock.

CHAPTER 7

7. DESILTING BASIN

7.1 Generalities

The conventional water intakes are designed 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, 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.

There are many systems of desilting. However, for the purpose of this report, the writers have selected only two systems, one for a run-of-river plants, and the other for drop intakes, selecting of course, desilting plant types that have proven to be advantageous in the range of discharges being considered.

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

The second one corresponds to desilting basins for drop intakes which operate with larger quantities of sediments and sand to be picked up in the intake; a different approach for this type is required (see Chapter 6, Section 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; mechanized systems are therefore 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 realized 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 immediately at the entrance by means of an entrance sill, diverting the sediments directly into the sluice channel, where they can be flushed downstream periodically.

Although the trash rack is designed to provide a maximum of protection against sediments, a small percentage of sediments carried by the river always manages to get into the entrance of the water ways, especially 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 provided. This is all the more important if the turbines are of the Francis or of the Pelton type; for Crossflow turbines the problem is somehow less critical.

In order to be able to ensure 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 have a grain diameter within a range between 0.2 and 0.5 mm.

$$\mathbf{0.2\ mm \leq diameter,\ d \leq 0.5\ mm}$$

According to Vischer and Huber, 1982, the critical water velocity can be estimated using the following formula :

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

which means that for water velocities of the order of 0.20 m/s be removed all particles with diameters greater than 0.20 mm but smaller than 0.50 mm.

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

Table 7.1 gives the approximate main dimensions of the desilting basin for conventional run-of-river intakes as a function of the design discharge Q_{design}:

TABLE 7.1
MAIN DIMENSIONS OF A CONVENTIONAL DESILTING PLANT

Q_{design} (m^3 / 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

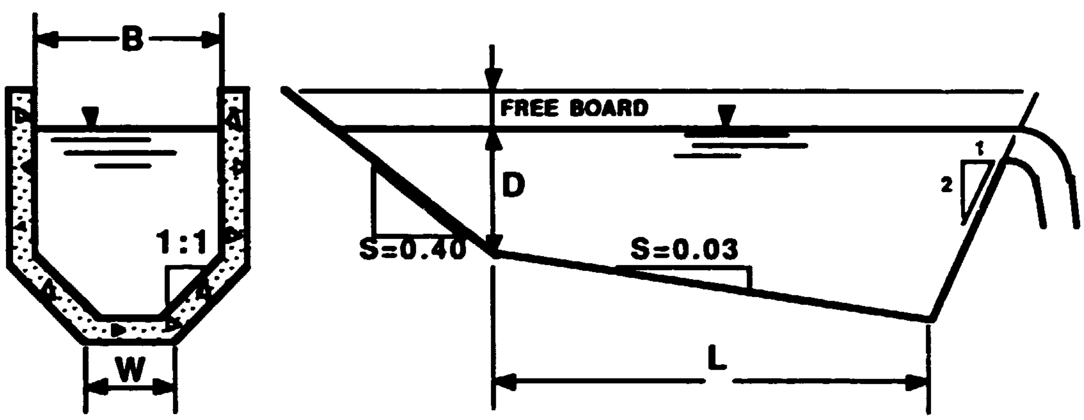


FIG. 7.1 Main dimensions of a conventional desilting basin.

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.

The depth of coarse sediments in front of the intake sill and sluice channel should be checked regularly to avoid their excessive accumulation

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

7.3 DESILTING BASINS FOR DROP INTAKES

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

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

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

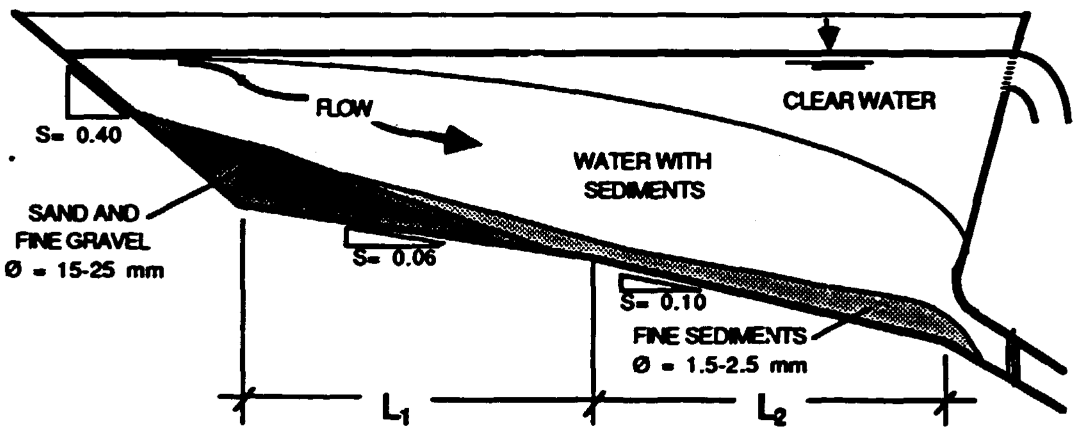


FIG. 7.2 Sedimentation process in the desilting basin

The following restrictions apply to this type:

- ◊ During flushing operation, the power plant should not be in operation
- ◊ The depth of sediment 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 section.

The main dimensions of desilting basin for drop intakes as a function of the design discharge Q_{design} are given in Table 7.2

TABLE 7.2 MAIN DIMENSIONS OF DESILTING BASINS FOR DROP INTAKES

Q_{design} (m^3/s)	B (m)	L_1 (m)	L_2 (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.

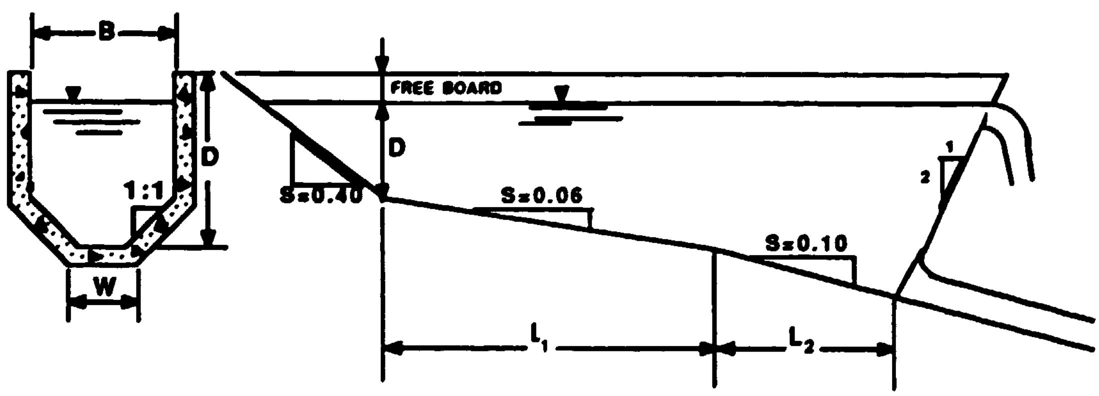


FIG 7.3 Main dimensions of a desilting basin for a drop intal

CHAPTER 8

8. ADDUCTION CANAL

It is important to control the amount of water that enters the adduction canal in order to avoid undesirable spilling of water over the canal sides.

At the entrance of the canal a side spillway should be provided 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 the in which 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 obviously more favourable and cheaper to close down 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 S = 0.0012 with a bottom of concrete type PC - 300 of 0.30 cm thickness lightly reinforced with steel bars. 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 excavated directly into the ground. This is generally not recommended 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 therefrom.

In the technical literature it is recommended not to exceed a water velocity of 0.75 m/s in canals without lining; Table 8.1 gives water velocities limits in canals built directly on various kinds of soil materials :

TABLE 8.1
MAXIMUM ALLOWABLE LIMITS OF WATER VELOCITIES IN CANALS TO PREVENT EROSION.

SOIL MATERIAL	LIMIT OF WATER VELOCITY (m/s)
Fine sand	0.40
Sandy clay	0.50
Clay, alluvial soils without cohesion	0.60
Ordinary clay, fine gravel	0.70
Alluvial soils with some cohesion or a mixture of gravel sand and clay	1.00
Gravel (Ø < 1 cm)	1.20
Gravel (1.0 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 a velocity, sediments begin to settle; they eventually obstruct the canal. For that reason it is recommended to choose a design with water velocities that are somewhat lower than the above allowable values. A factor of safety of the order of three should be the rule, but on the other hand, the velocities should be kept high enough to avoid a sedimentation in 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:

$$V_s = C \cdot t^{0.64} \tag{8.1}$$

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 absence of definite values Table 8.2 gives as a guideline some limit velocities for 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

Usually applied canal sections for small hydropower schemes are indicated on Fig. 8.1 and on Table 8.3 in function of the design discharge:

TABLE 8.3 MAIN DIMENSIONS OF CANALS

Q_{design} (m^3/s)	V (water) (m/s)	B_1 (m)	B_2 (m)	W (m)	H (m)	RADIUS IN CURVES (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

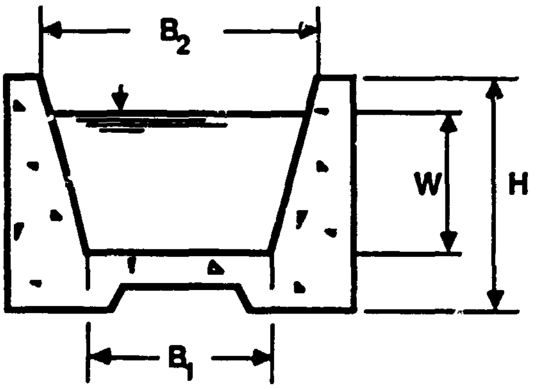


FIG. 8.1 Typical canal sections for small hydropower projects

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

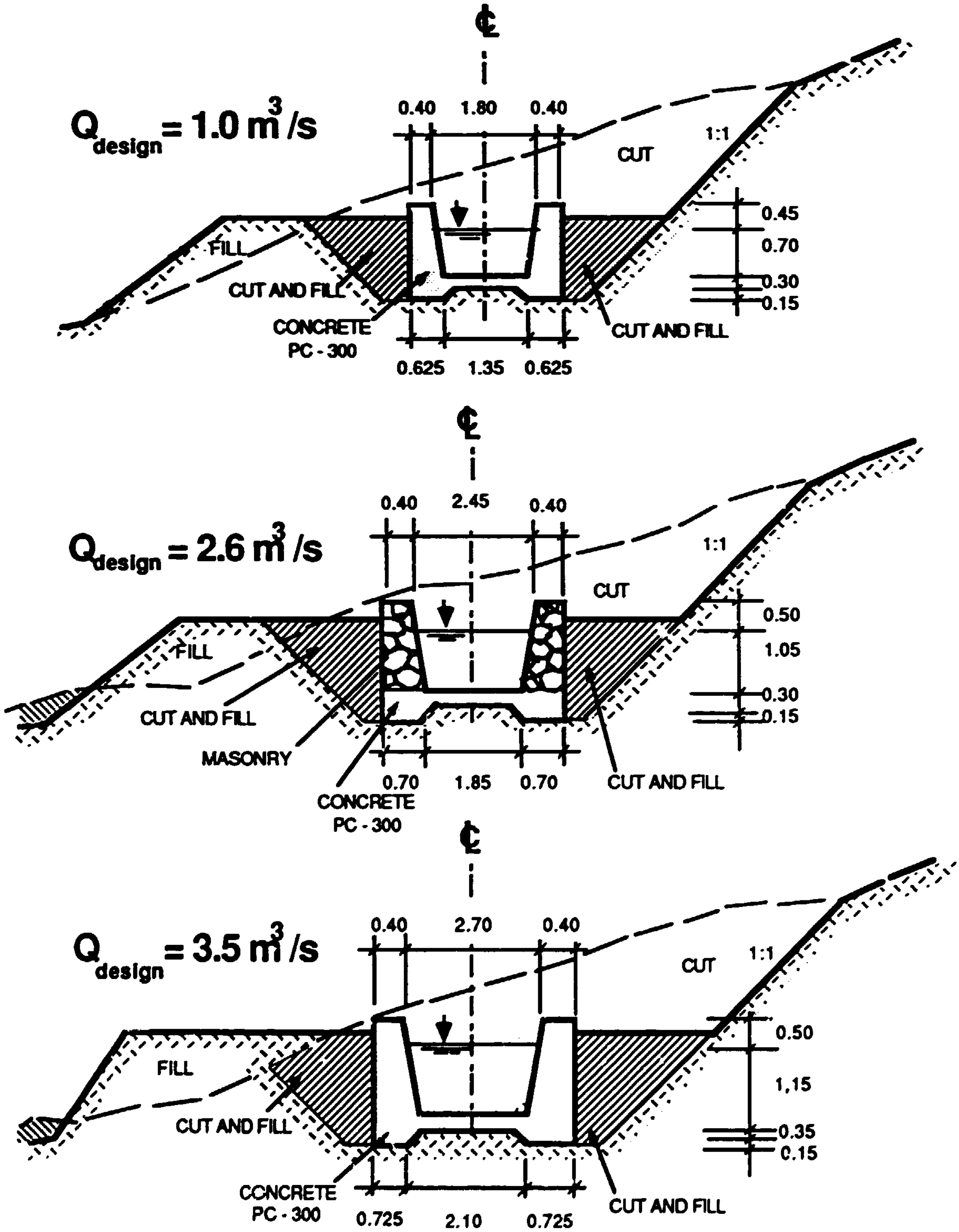


FIG. 8.2 Examples of Adduction Canals with Slope S ≈ 0.0012

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 guarantees not only the conduction of water free of air, but also 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 fulfil 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 of low pressure.

For small plants, the surface area of forebays required can be estimated as a function of the maximum discharge with the rule of thumb as shown in Table 9.1

TABLE 9.1
RECOMMENDED FOREBAY AREA FOR SMALL HYDROPOWER PLANTS

Discharge Q (m^3/s)	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 mentioned above 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 exceeds 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 which have been selected as prototypes in this report present most benefits and positive performance.

CHAPTER 10

10. PENSTOCK

10.1 Generalities

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

Its design should be aimed at maintaining the **L/H** relation (length to height) as small as possible; consequently, the penstock should be as perpendicular as the topographical conditions will allow. This assumes that stable geological conditions prevail allowing the penstock to be anchored on solid rock.

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

Depending on the head of the scheme, the size of the penstock and the surrounding conditions, the material of the penstock can vary. In some cases for low and medium heads and short distances, a simple Polyvinyl Chloride pipe (PVC), or concrete pipes can be used, However, steel pipes are as a rule more suitable; they are indispensable for medium and high heads and longer distances (*penstocks built with PVC are usually buried into the ground because, if exposed, the ultraviolet radiation of the sun will reduce their life time rapidly*).

For the above mentioned reasons, the most commonly used and recommended material for penstocks pipe is steel, which offers the highest technical and maintenance advantages.

The penstock can be installed above ground (non embedded), buried in the ground, depending on the natural soil conditions, or buried in concrete and tunnel liners. However, for small hydropower schemes, the above ground or the buried in the ground systems are commonly used, other systems being used very seldom. For obvious economical reasons, the "above ground systems" are preferred; they are easy to check and to maintain, but in some countries are more susceptible to acts of sabotage, which today is an additional aspect to consider during the design stage.

It would take considerably more time and space to present each and every possible type of penstock.

Although, as mentioned above, the penstock can be made of PVC, steel pipe is required for most schemes, and therefore solid anchor blocks have to be provided to ensure that the resisting forces will not displace and damage the rigid pipe. Support conditions for these two alternatives are obviously very different.

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

When the penstock exceeds certain lengths, expansion joints have to be provided 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.

Penstocks for small hydropower stations are usually of the open air type, however, sometimes for physical or economical reasons, the buried type is preferred.

Buried penstocks require a different approach, and further on, we will examine their main features, without entering into details.

The reason which leads to the choice of a buried steel conduit are principally:

a) If the conduit passes across parts of soil with low admissible pressure. This type of construction gives a low uniform pressure distribution on the whole length of the steel conduit.

b) When the steel conduit passes through cultivated land.

c) When the steel conduit passes through a forest and/or steep areas

d) In areas in which for ecological or environmental reasons the surroundings should not be disturbed, the steel conduit will be completely buried, so that after completion of the works the terrain surface will be restored to its original condition.

The alignment of the steel conduit should be adapted to the topographical conditions, however, it must preferably be designed with long straight parts, in plan as well as in elevation

The steel conduits of buried penstocks are completely welded; each tube, generally of 6 m length, will be welded, end by end on the job site, including the bends. (those bends are however, previously manufactured with pipe segments in a suitable work shop).

The longitudinal tensions are taken-up and distributed by a sand cushion laid at bottom of the trench, (See Example 9, Drawing 40093-9-4)

Normally anchor blocks are not necessary except before the entry in the power house. It is recommended to embed in concrete all bends with an angle of more than 30°.

The conduit forms a long "tube" continuous and without expansions joints (which are more expensive). Considering that the conduit after its erection and welding will be covered with 50 - 60 cm of earth fill, the buried conduit will remain at a constant temperature equal to the outside temperature of the surrounding ground material.

Similarly to other types of penstocks, the geological conditions must be well studied and evaluated; areas with hazardous parts and possible sliding conditions should be avoided, by-passed or contourned. At road crossings with traffic, rivers, creeks etc., the conduit is usually buried in concrete for protection.

10.2. Penstock location

The location and disposition of a penstock depends upon the topographical and geological conditions, the location of the water intake and desilting structures, the outlet works, the relative location of the power house, and upon 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 single penstock with a dividing system to feed all units is governed by the economics and by the necessary flexibility of operation.

This report will consider the single penstock system only.

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; final design will also take into account availability of suitable manufactured components

10.3 Sizing of the Penstock

The minimum water velocity to be used in the design of penstocks for small hydro power plants is a function of the relation L/H, 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

These values are taken from the experience obtained in many projects (Values that are valid only for small power plants). As a general rule the maximum water velocity should not exceed 6 m/s.

Knowing the L/H relation and the minimum and maximum water velocities allowed, a first approximate diameter can be chosen in function of the design discharge.

10.4 Head losses in Penstocks

To calculate the hydraulic energy line, it is necessary to determine the head losses in the pipe. 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 the 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 guide vanes.

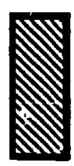
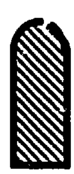
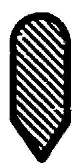
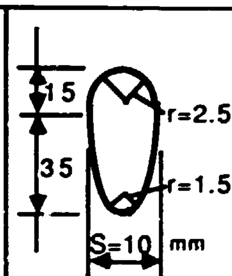
The **losses at the trash rack** for the range of values under study in this report are:

$$h_1 = K_1 \frac{v^2}{2g} \sin\alpha \qquad (10.1)$$

where:

$$K_1 = \beta \sqrt[3]{\left(\frac{s}{b}\right)^4} \qquad (10.2)$$

α, ß, s and b as depicted in Fig. 10.1

FORM					
VALUE OF β	2.40	1.80	1.04	1.07	0.76

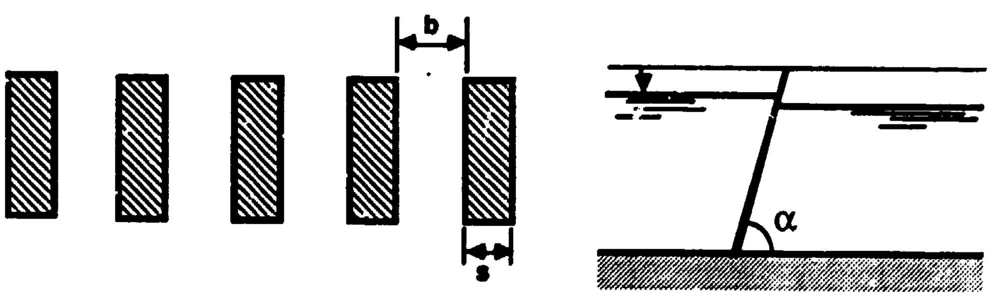


Fig. 10.1 Coefficient K_1 of losses at the Trash Rack

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

$$h_2 = K_2 \frac{v^2}{2g} \tag{10.3}$$

where K_2 can be obtained from figure 10.2

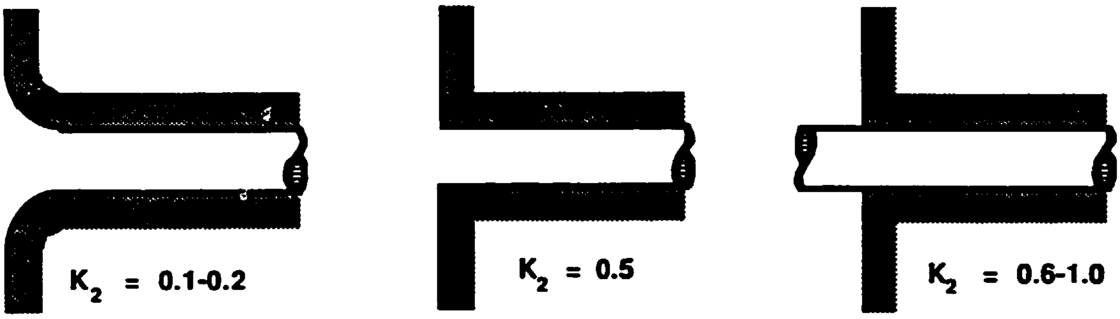


Fig. 10.2 Coefficient K_2 of losses at the entrance

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

$$h_3 = \frac{v^2 L}{K^2 R^{\frac{4}{3}}} \tag{10.4}$$

where: K = Strickler Coefficient of Friction
- K = 80 for concrete
- K = 90 for steel
- K = 100 for PVC

L = Length of pipe in m
R = Hydraulic radius
v = velocity of water in m/s

Note: The hydraulic radius R being defined as the <u>wet</u> sectional area of the pipe divided by the <u>wet</u> perimeter

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

$$h_4 = K_4 \frac{v^2}{2g} \tag{10. 5}$$

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

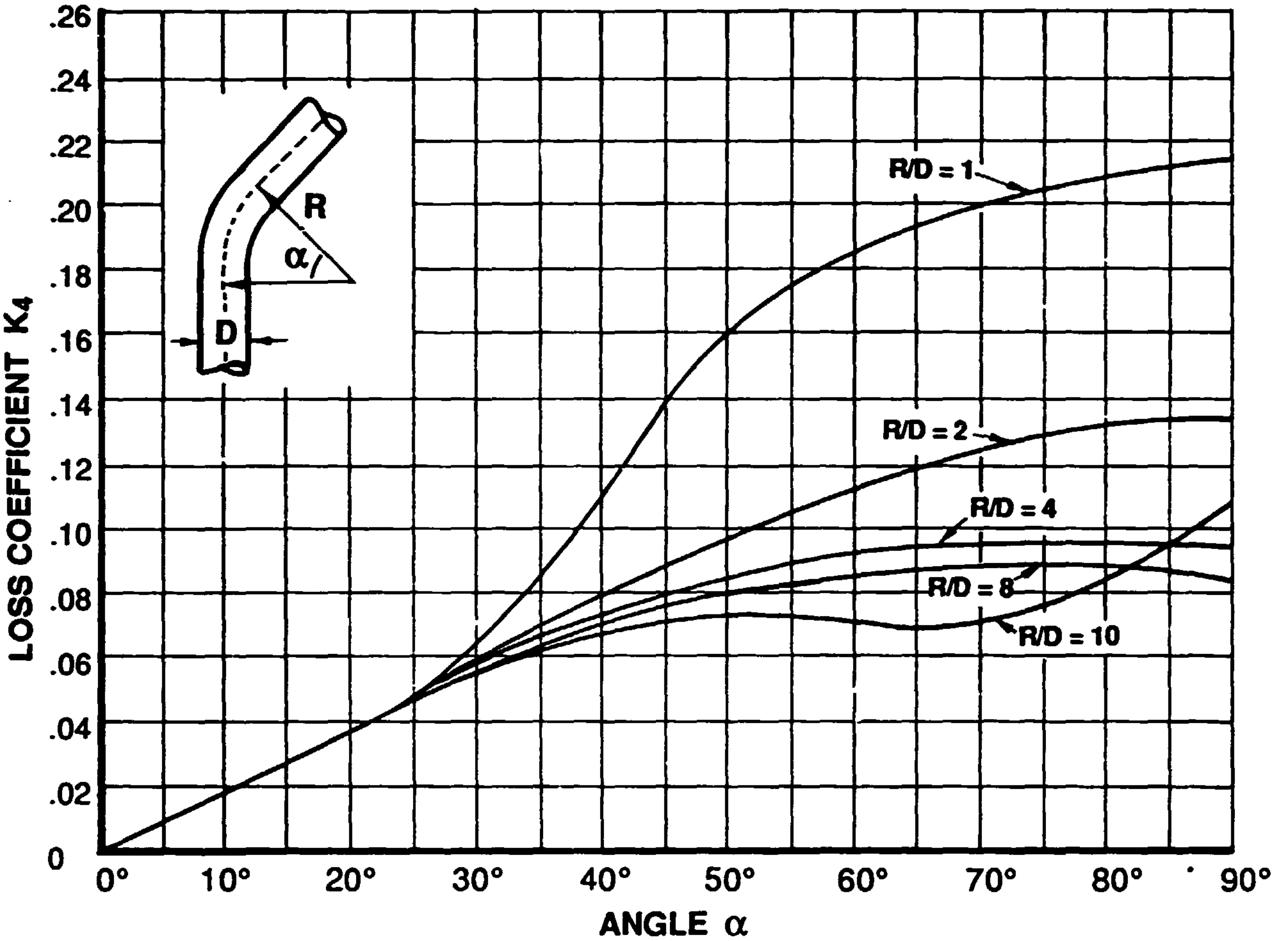


Fig. 10.3 Coefficient K_4 of losses due to pipe bends

The losses at Gate valves and regulating guide vanes are:

$$h_5 = K_5 \frac{v^2}{2g} \tag{10.6}$$

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

Table 10.2 Values of the Coefficient K_5 for Gate Valves

D (m)	% of Gate Valve Opening				
	10	25	50	75	100
0.30	56.00	12.00	2.50	0.50	0.07
0.50	48.00	10.30	2.14	0.40	0.06
1.00	40.00	8.57	1.79	0.34	0.05
2.00	32.00	6.80	1.43	0.27	0.04

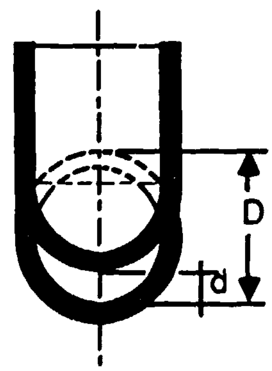
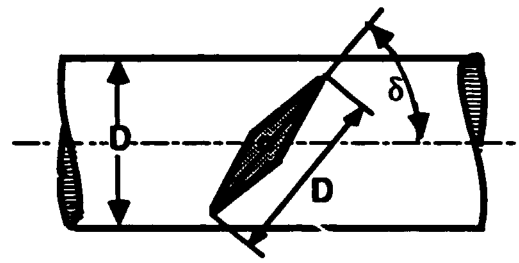


Table 10.3 Values of the Coefficient K_5 for Butterfly Valves

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 The Waterhammer Phenomenon

Waterhammer may occur in full-flowing penstocks when either the flow is retarded or accelerated; opening or closing the penstock valve will create such conditions.

As long as the above changes (opening and closing) are controlled and gradual, conventional surge calculations can be applied, whereby water is considered as incompressible and the penstock rigid.

However, quick closing of the penstock-valve, will reduce the flow of water rapidly, causing an increased head to be generated upstreams; consequently a pressure wave will be propagated upstream of the penstock, which will decrease the velocity of the flow. Conversely the pressure will be reduced in the downstream flow of the valve, with a resultant lower pressure wave traveling downstreams, and a corresponding reduction of flow velocity.

Fig. 10.4 describes graphically the sequence of events following a sudden valve-closure at the bottom of the penstock, whereby friction between water and inner pipe surface has been neglected. What happens is the following:

A short instant ($t+\varepsilon$) after the valve has been closed the water nearest to the valve is compressed and brought to rest; the pipe walls are thereby stretched. As soon as this "stretching" in the lower layers is completed, the process is repeated successively in the next higher layers. Upstream water will however move with undiminished speed until all layers have been compressed up to the forebay. Higher pressure moves upstream as a wave, bringing the water to rest as it proceeds, compressing it and stretching the penstock.

As soon as the wave reaches the forebay, all water in the penstock is subjected to a supplementary head Δh; at that time all movement stops the kinetic energy having been converted into elastic ("stretched") energy. (See Fig. 10.4B)

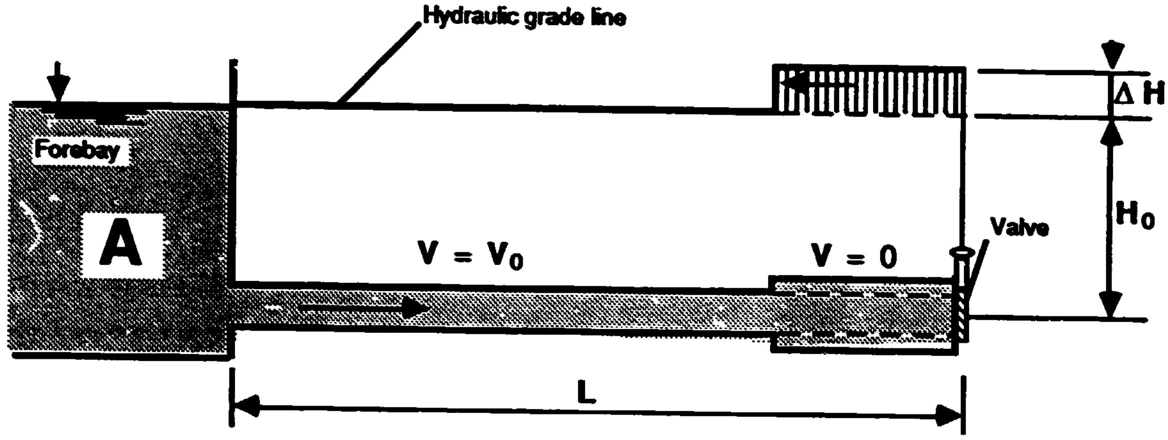
An unbalanced condition occurs as soon as the above pressure wave arrives at the upper end of the penstock, because the forebay (reservoir) pressure remains unchanged. At this very moment, beginning at the upper end of the penstock, the water starts to flow backwards. Its pressure reverts to that value which applied before the water closure, the penstock returning to its previous condition; the water velocity becomes $-V_0$ at the time $t = L/a + \varepsilon$ (See Fig. 10.4 C); The reverse process -the pressure wavemoving downwards- takes place with the speed of sound (in the water). At the time $t = 2L/a$ the wave arrives at the valve (See Fig. 10.4D) with equal velocities $-V_0$ prevailing all along the penstock.

Because the valve is closed and no fluid is available to maintain an exit flow, a low pressure develops ($-\Delta h$) such that the fluid is brought to rest. This low pressure wave travels upstream at a speed a bringing everywhere the water to rest; at the same time the water expands because of the lower pressure, and allows the penstock walls to contract, (see Fig. 10.4 E).

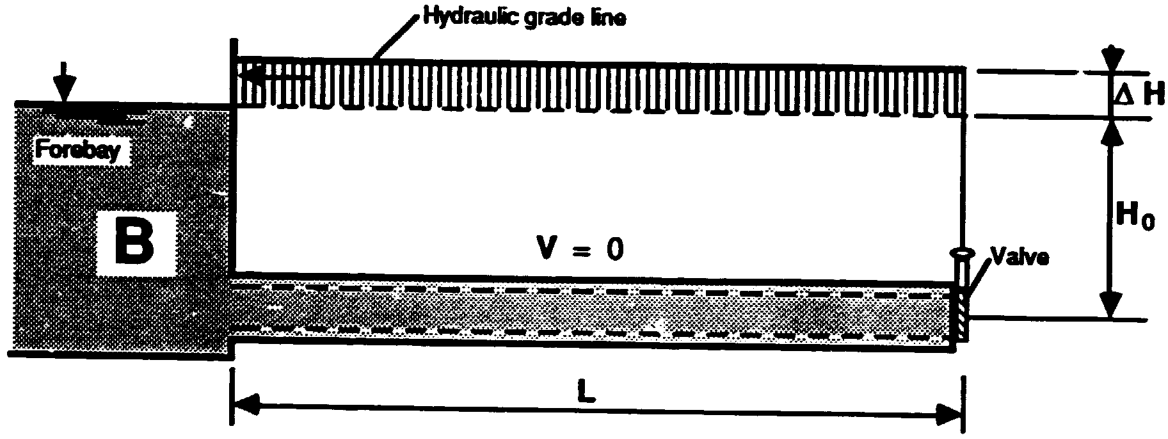
As soon as the low pressure wave arrives at the top end of the penstock which takes place at a time $t = 3L/a$ seconds after closure of the valve, the water is at rest, however with a pressure $-\Delta h$ less than before closure of the valve (See Fig 10.4F). This causes a renewed unbalanced condition at the forebay end of the penstock, resulting in a renewed water flow into the penstock with a velocity V_0 downwards; water and penstock return to normal conditions as the wave travels downstream at the speed a (See Fig. 10.4G). When the wave reaches the valve, conditions are exactly the same as when the valve was closed, $t = 4L/a$ seconds earlier.

The process would be repeated indefinitely in the following 4L/a second periods, if the friction in the penstock, and imperfect elasticity of water and penstock walls were not taken into account. Eventually both above factors bring the water to a permanent rest.

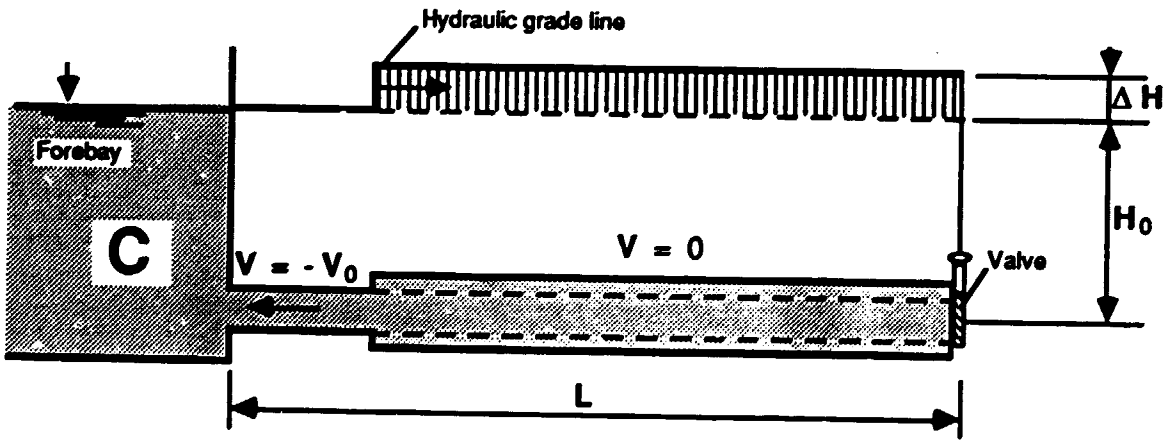
Valve closures occurring in times less than 2L/a seconds are called **rapid closures**; closure times above 2L/a seconds are **slow closures**.



CONDITIONS AT $t + \varepsilon$

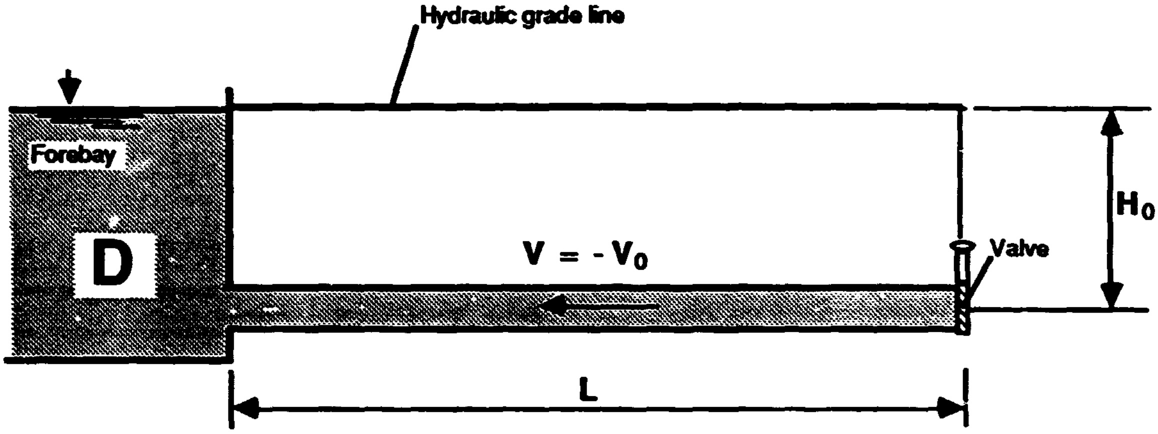


CONDITIONS AT $t = \frac{L}{a}$

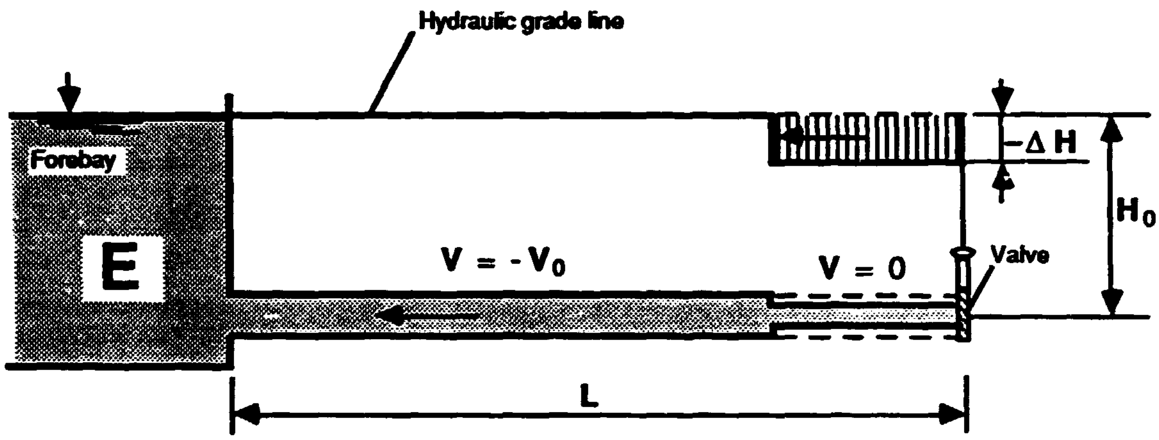


CONDITIONS AT $t = \frac{L}{a} + \varepsilon$

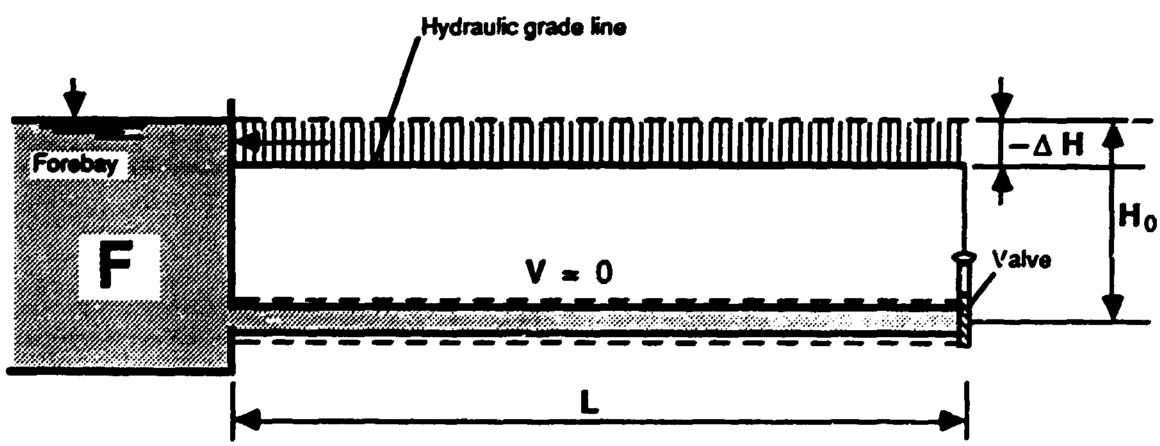
Fig. 10.4 The Waterhammer Phenomenon



CONDITIONS AT $t = \frac{2T}{L}$

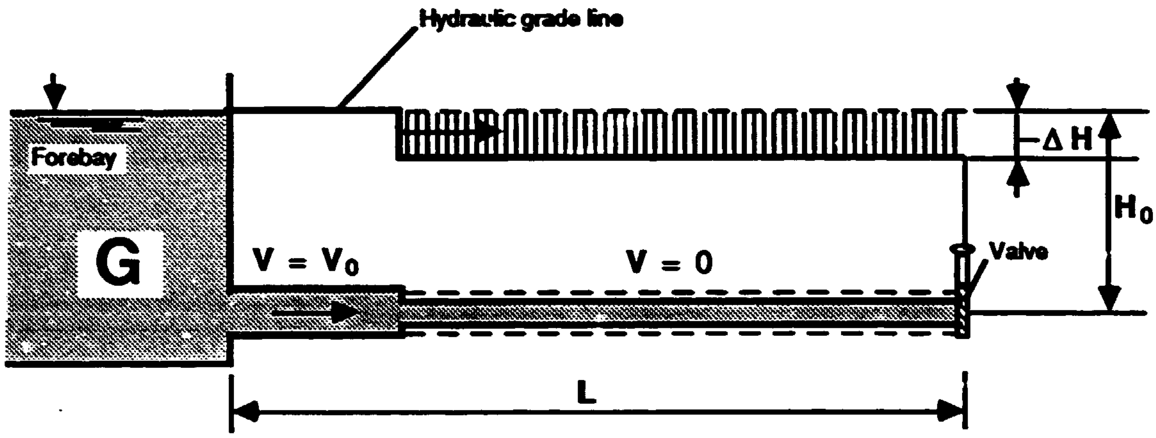


CONDITIONS AT $t = \frac{2L}{a} + \varepsilon$

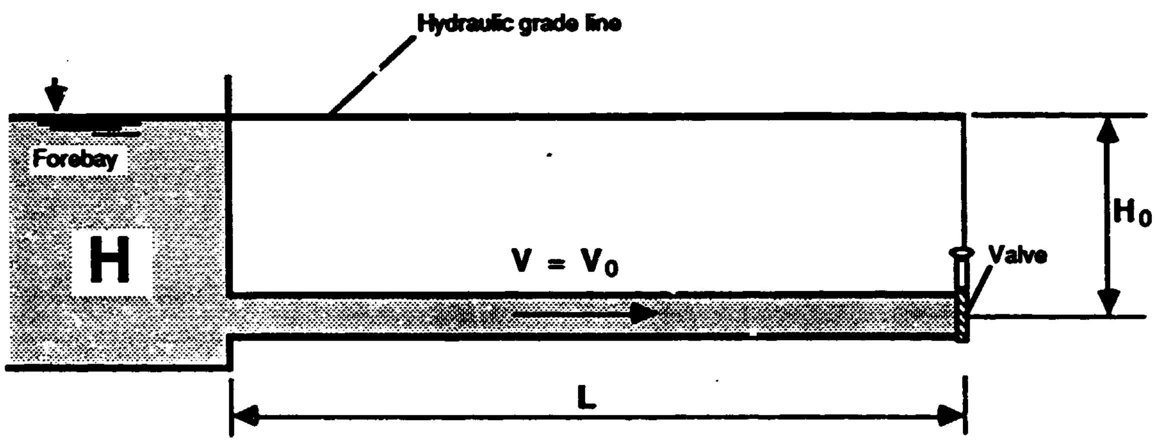


CONDITIONS AT $t = \frac{3L}{a}$

Fig. 10.4 The Waterhammer Phenomenon (cont'.)



CONDITIONS AT $t = \frac{3L}{a} + \varepsilon$



CONDITIONS AT $t = \frac{4L}{a}$

Fig. 10.4 The Waterhammer Phenomenon (cont'.)

10.6 Effects of Water Hammer

As we have seen, 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 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:

$$T_s = \frac{v\,L}{g\,H} \qquad (10.7)$$

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:

$$\Delta H = \frac{a\,v}{g} \qquad (10.8)$$

where:

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

To calculate the value of ΔH, it is necessary to determine the value of the pressure wavespeed a.

10.7 The wavespeed "a"

At the moment that the valve at the lower end of a penstock is closed, the water immediately adjacent to it is brought from the velocity V_0 to rest by the impulse of the higher pressure developed at the inlet face of the valve, a process that continuously develops upstream of the valve, and which can be visualized in the form of a pulse wave of higher pressure moving with a sonic wavespeed "a".

It is pertinent to mention , that the wavespeed "a", depends also on the configuration and the system of anchorage used in the penstock; the wavespeed a is different in the case of a penstock anchored only in its upper part and free to move in the longitudinal direction downstream, or when the penstock is welded and firmly anchored along its longitudinal axis, or lastly when the pipe includes expansion joints, allowing differential longitudinal movements.

By application of the continuity equation a numerical value of "a" can be calculated. In this report only three support situations for a thin-walled penstock are considered; they are the three most frequently cases encountered in actual practice i.e.:

(a) Penstock anchored against longitudinal movement throughout its length
(b) Penstock anchored against longitudinal movement at the upper end
(c) Penstock anchored with frequent expansion joints

(Note: For other cases, consult the respective technical literature, like Streeter and Wylie, 1978, or M.H. Chaudhry, 1979)

The general expression for the wave velocity as presented by Halliwell can be written as:

$$a = \sqrt{\frac{K}{\rho\left[1 + (K/E)\,\psi\right]}} \qquad (10.9)$$

in which ψ is a non dimensional parameter that depends upon the elastic properties of the conduit:

E = Young's modulus of elasticity of the penstock walls (See Table 10.4)
K = the bulk modulus of elasticity of water (See Table 10.5)
ρ = the density of water (See Table 10.5)

For thin-walled elastic conduits, the value of ψ is respectively:

(a) For penstocks of small hydro power plants anchored against longitudinal movement throughout their length

$$\psi = \frac{D}{e}\,(1 - \nu^2) \qquad (10.10)$$

(b) For penstocks of small hydro power plants anchored against longitudinal movement at the upper end:

$$\psi = \frac{D}{e}\,(1.25 - \nu\ ) \qquad (10.11)$$

(c) For penstocks of small hydro power plants anchored with frequent expansion joints:

$$\psi = \frac{D}{e} \qquad (10.12)$$

Formulae in which D = penstock internal diameter,
e = wall thickness and
v = Poisson's ratio (See Table 10.4)

Assuming first the value of the wall thickness e, the parameter ψ and the wavespeed a can be calculated; the thickness of the pipe shell is then computed and compared with the assumed value, iterating the procedure until a final value for the wall thickness is obtained. This value should be rounded off to the next higher size of commercial thickness according to manufacturers or suppliers catalogues.

TABLE 10.4

Young's Modulus of Elasticity and Poisson's Ratio for some Penstock Material of Small Hydro Power Schemes

Material	Modulus of Elasticity E (Gpa)	Poisson's Ratio (v)
Cast Iron	70 - 160	0.25
Concrete	15 - 30	0.10 - 0.15
Mild Steel	200 - 210	0.27
PVC Rigid	2.40 - 2.70	0.40

TABLE 10.5

Bulk Modulus of Elasticity and Density of Water at Atmospheric Pressure

Liquid	Temperature (°C)	Density ρ (kg/m3)	Bulk Modulus of Elasticity, K (GPa)
Water, fresh	20	999	2.19
Water, sea	15	1025	2.27

Example: For conducting a transient analysis, the waterhammer wave velocity in a penstock for a small hydropower scheme should be computed, assuming first that that the penstock is anchored against longitudinal movement throughout its length, second, that it is anchored against longitudinal movement only at its upper end, and third that the penstock is provided with sufficient expansion joints

Assuming that the penstock of mild steel is 50 m long with a 1.20 m diameter and of 20 mm wall-thickness, the following cases can be investigated:

Case 1: The penstock is anchored along its length:

$$\psi = \frac{D}{e}(1 - \nu^2)$$

$$\psi = \frac{1.20}{0.02}(1.25 - 0.27^2) = 70.63$$

$$a = \sqrt{\frac{K}{\rho\left[1 + (K/E)\,\psi\right]}}$$

from Table 10.1 K = 2.10 GPA and from Table 10.2 E = 210 GPa and ρ = 999 kg/m^3

$$a = \sqrt{\frac{2.19 \times 10^9}{999\left[1 + (0.0104 \times 70.63)\right]}}$$

$$\underline{a = 1124 \text{ m/sec}}$$

Case 2: The penstock is anchored at the upper end only

$$\Psi = \frac{D}{e}(1.25 - \nu\ \ )$$

$$\Psi = \frac{1.20}{0.02}(1.25 - 0.27) = 58.80$$

$$a = \sqrt{\frac{2.19 \times 10^{9}}{999\left[1 + (0.0104 \times 58.80)\right]}}$$

$$\underline{a = 1166\ \mathrm{m/sec}}$$

Case 3: The penstock has sufficient expansion joints:

$$\Psi = \frac{D}{e}$$

$$\Psi = \frac{D}{e} = \frac{1.20\ \mathrm{m}}{0.02\ \mathrm{m}} = 60$$

$$a = \sqrt{\frac{2.19 \times 10^{9}}{999\left[1 + (0.0104 \times 60)\right]}}$$

$$\underline{a = 1161.80\ \mathrm{m/sec}}$$

Note: Although the above 3 results for "a" are similar, it is pointed out that in practice very different values can be obtained of the three formula

Given the diameter of the pipe, the water velocity can be evaluated, and consequently the pressure wave speed can be calculated as previously indicated; finally the value of the water hammer pressure ΔH can be estimated

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 determined and the pipe thickness calculated and designed to take care of the resultant total head.

Surge tanks are usually connected to the penstock to reduce the water hammer effects.

10.8 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 bending stresses, in addition to the internal stresses.

The plate thickness of penstocks of small hydro schemes can be computed with the following formula:

$$e = \frac{D\,p}{2\,S\,\varepsilon} + 3 \qquad (10.13)$$

where:
- S = Hoop tension in kg/cm^2
- D = Inside diameter in mm
- p = internal pressure in kg/cm^2
- e = plate thickness in mm
- ε = efficiency of joint ASTM stipulates:70-100% for double welded and 65-90 % for single welded.
- 3 = 3 mm additional thickness as corrosion allowance.

Depending whether the penstock is of the surface or embedded structure type, the design conditions will be different.

10.9 Design conditions for surface penstocks

For a surface penstock design, it is recommended to consider the following conditions and allowable stresses for its body shell:

1. The normal condition, is based upon a maximum static head plus the pressure rise due to the water hammer (aprox. 15% of the H_{gross}).

The allowable stresses to be used in the design should be equal to 2/3 of the specified minimum yield stress or 1/3 of the minimum specified tensile strength, choosing whichever is the smaller.

2. The intermittent condition, take into account the following situations:

a) Pressure test during the filling of the penstock.

b) Draining of the penstock
c) Earthquake occurrence during normal operation.

For the three situations mentioned above, the allowable stresses to be used equal 80% of the specified minimum yield stress, or 50% of the specified minimum tensile strength, choosing whichever is the smaller value.

3. The emergency case, which takes into account situations like partial gate closure in a time of 2 L / a seconds at a maximum rate of operation with an inoperative governor of the turbine at the same time,

For this condition, the recommended allowable stress is equal to 2/3 of the minimum specified tensile strength, but in no case shall the allowable stress exceed the specified minimum yield stress.

The most common allowable design stresses in the shell for the above mentioned design conditions are given in table 10.6 , for special cases in which material with higher properties will required, the designer should consult the local producers or manufacturers.

The above stresses should not be exceeded in any case, in any part of the shell, even after combining the longitudinal and circumferential stresses.

In spite of being a surface penstock, it should be designed, to withstand external pressures with a factor of safety of 1.5.

10.10 Buried Penstocks

For the case of buried penstocks in normal soil conditions of trench and fill, the design should consider the following situations:

1) The design for the internal pressure should be based on the same three conditions as mentioned for the surface penstocks.

2) The determination of the wall thickness made using the following empiric equations:

a) for penstocks with diameters up to 1500 mm:

$$e\,(mm) = \frac{D\,(mm)}{288} + 3\,(mm) \qquad (10.14)$$

b) for penstocks with diameters of 1500 mm or greater:

$$e\,(\text{mm}) = \frac{D\,(\text{mm}) + 508}{400} + 3\,(\text{mm}) \qquad (10.15)$$

c) in any case the thickness should be the greater of a) or b) but at the same time, greater that the e_{min} of:

$$e_{min} \geq 6.35\ \text{mm}\ (1/4\ \text{inch}) \qquad (10.16)$$

where: D is the penstock diameter in mm and the value of 3 mm added to the formula is an additional security against corrosion.

The buried penstock should be designed to withstand external pressures with a safety factor of at least 1.5 which should be included in the load factors to be used.

In Table 10.6 Allowable stresses are given for the most common cases of penstocks made in steel.

TABLE 10.6 Materials and Allowable Stresses

ASTM Designation	Minimum tensile strength kg/cm2	Minimum yield stress kg/cm2	Allowable stresses, kg/cm2		
			1. Condition Normal	2. Condition Intermittent	3. Condition Emergency
A285					
Grade A	3164	1687	1054	1350	1687
Grade B	3515	1898	1167	1518	1898
Grade C	3867	2109	1286	1687	2109
A515 / A516					
Grade 60	4218	2250	1406	1800	2350
Grade 65	4570	2460	1518	1968	2460
A516					
Grade 70	4921	2671	1638	2137	2671

Source: Bureau of Reclamation USA.

10.11 Civil works for buried penstocks

According to the longitudinal profile and the importance of the conduit, a rail track should be built parallel to the penstock. For transversal access and small transportation needs, tracks to some parts of the pipe will be built. From those points the tubes will be lowered parallel to the tracé with the help of a winch, rails and chariots. An area for pipe handling should be provided which will also serve as a temporary deposit of the pipes.

The main phases of the civil works are:

10.11.1 Trench:

- A geodetic longitudinal profil must be prepared to be used for the accurate design of the conduit.
- Deforestation (all roots and trees must be cut: a contact of the roots with the pipe should not be allowed).

- Where doubts exist, because of possible influence of the trees, it is recommended to bury the conduit in concrete

- Wherever possible the top soil should be removed, and deposited carefully in strips along the side of the trench. Obviously the strip material should be held in position by means of wattling, planks, etc.

- Provide a proper drainage of rain and filtration water.

- Shoring and bearing of the side walls of the trench where necessary

- Keep a continuous careful maintenance of the open trench

- Prepare in advance the joints required recesses on the job site, to facilitate the welding and isolation finishings during erection of the conduit.

- For a successful erection of the pipes it is important to have clean trench bottoms.

10.11.2 Placing of the welded steel conduit and filling of the trench

- The conduit (in tubes of a length of generally 6 m) will be placed in the trench on jute bags filled with approx. 40 liters of earth, the bags being placed in advance at distances of about 3 m on the bottom of the trench.

- if the bottom of the trench is rock, previous placing of a sand layer (d = 0-4 mm) with a thickness of about 10 cm is recommended.

- The pipes should be aligned one by one along the side of the trench

- To lower the pipes into the trench, right dress, welding, completing isolation by the joints. Sometimes it is possible to weld some pipes together outside the trench, before lowering those parts into the trench.

- Use concrete anchor blocks at all the bends

- As a temporary measure the pipe will be held in position with the help of weight riders of granular material with a diameter not larger than 50 mm (using one rider every 6 m); joints should stay free for checking during the pressure test, (See Fig. 10.5)

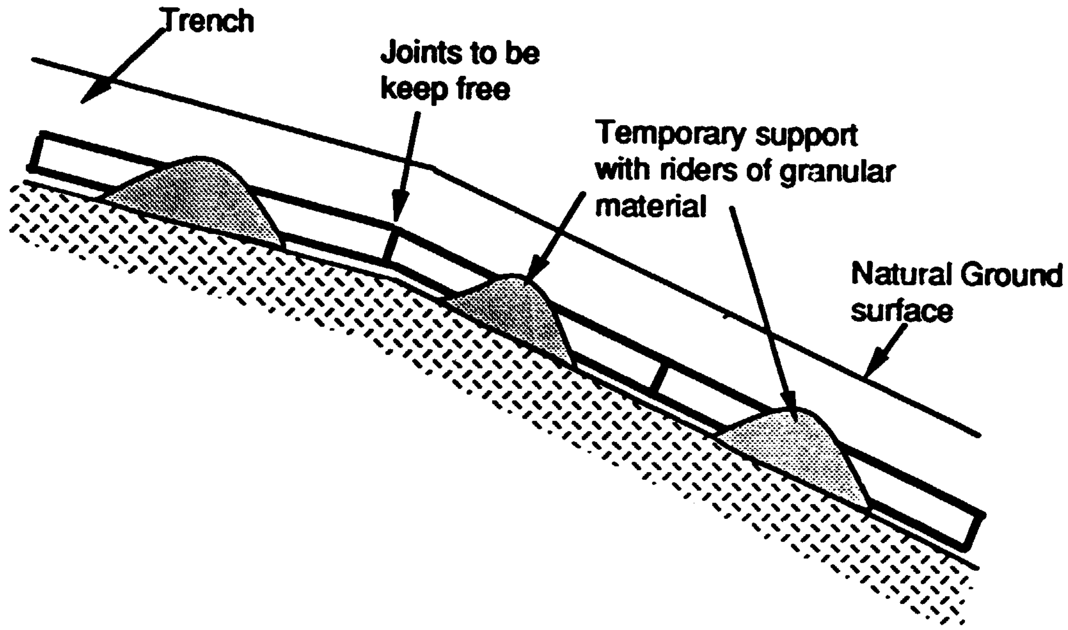


Fig. 10.5 Temporary fixation of the pipe with weight riders

- After the successful pressure test,(see example with the description of the pressure test), fill with soil material or sand gravel with particle diameters of 0 - 50 mm up to a height of 25 cm above the conduit. For the protection of the pipe, sharp edge stones should be avoided; in any case a direct contact of those sharp stones with the pipe surface should not be allowed.

- The backfilling of the trench shall be carried out in horizontal layers of 25 - 30 cm thickness, carefully compacted with a light compactor (a jumping rammer or a vibratory plate). The contractor of the Civil Works should give a good filling particular care, tamping and wadding it below and on the sides of the pipes. Filling of the upper part of the trench can be made with unsorted material. Wherever necessary transversal retention walls or locally obtained stone protection should be provided.

10.12 Welded Steel Conduit

The pipes for the welded steel conduit are to be manufactured by the suppliers in normal lengths of 6.00 m (if required they could be made in a length up to 12.00 m). Corrosion protection should be carried out at manufacturers' works, leaving only the last 20 cm of the ends free for welding on the job site.

The erection work should be made by experienced personel, taking especial care that the end to end welding of the pipes be made by a professional and experienced welder.

The steel quality of the pipe material should be FINE - GRAINED GRADE weldable.

Steel designation:

- FG 26 - DIN 1.0461 or
- DIN Standard WSt E 26 DIN Material No. 1.0462 or
- R ST 37 or similar

- The minimum yield point should be of σ = 2 555 kg/cm^2
- The allowable tension σ_{all} = 1400 kg/cm^2
- Normal pipe length 6.00 m

See example 9 for more details on pipe welding

10.13. Expansion Joints

Penstocks laid underground are affected by the temperature of the conveyed water and the temperature of the surrounding soil. If the penstocks are above the ground they 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 two fitting sleeves, one on top of the other prevent any leakage and enables longitudinal movements..

A typical expansion joint of this type is shown in figure 10.6

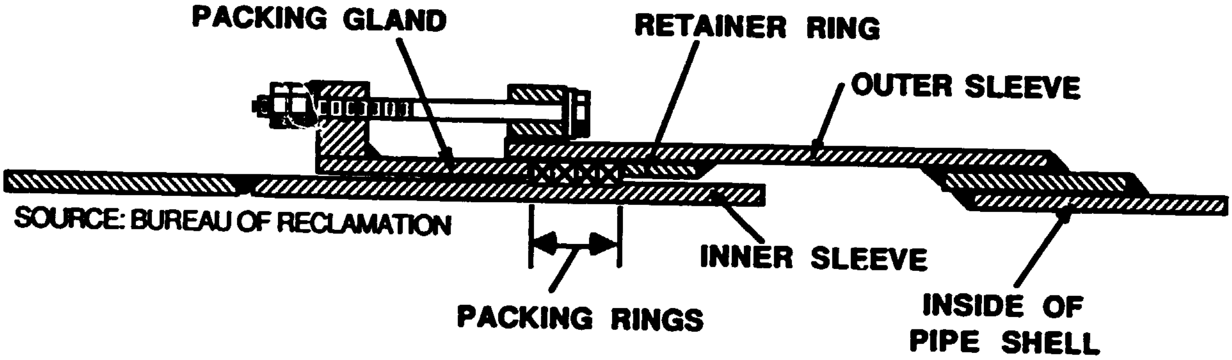


Fig. Nr 10.6 Sleeve type expansion joint

This This type of joint can be designed with two stuffing sides to allow transversal deflections and temperature dilatation movements, see figure 10.7.

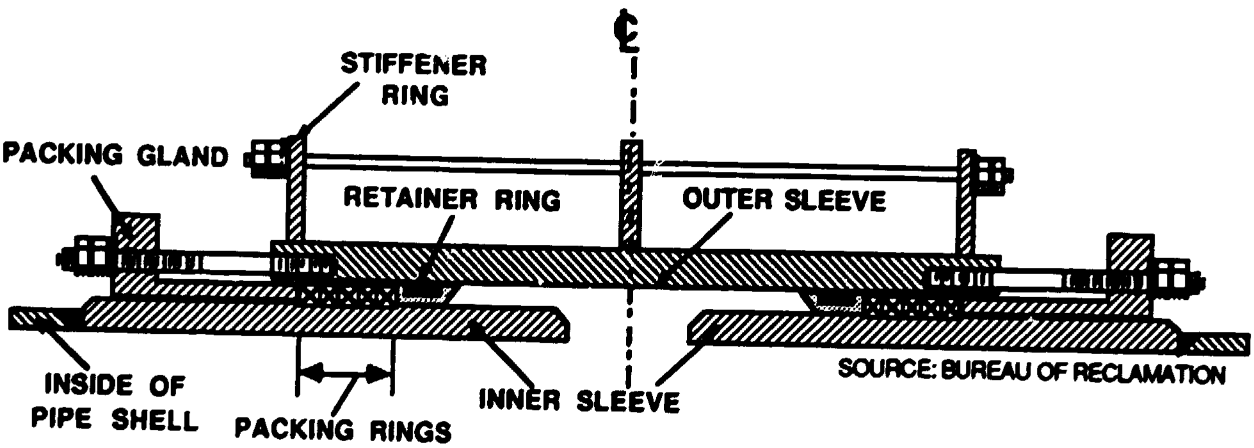


Fig. 10.7 Double sleeve type expansion joint

Another penstock system uses Dresser Coupling expansions, which are a patented; they allow an easy erection with a factory-made stress-free joint. With this type of sleeve locked up stresses otherwise caused by field-welded joints, are inexistent. Installation time can be sped up considerably.

In figure 10.8 a schematic view of a Dresser coupling is shown.

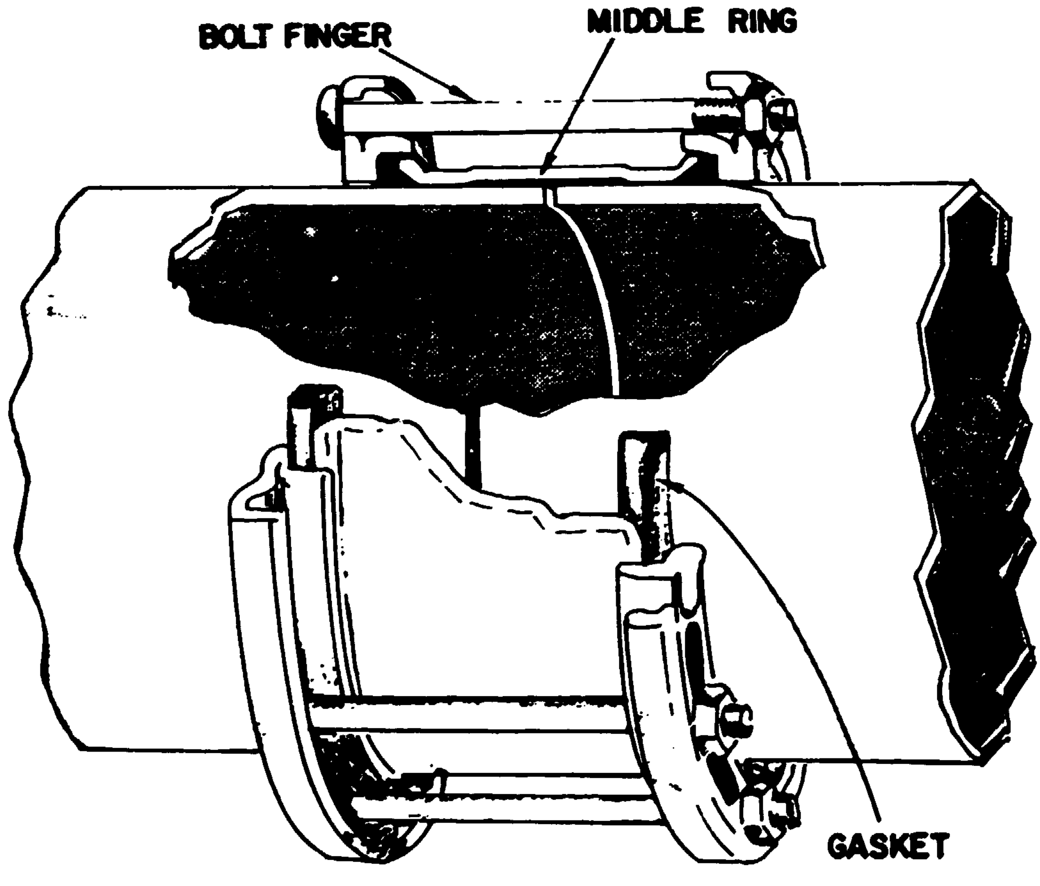


Fig. 10.8 Schematic view of a Dresser coupling

10.14 Anchors

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

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. However, the forces active on a pipe bend when the line is in operation mainly consist of:

- ◊ Temperature forces
- ◊ Hydrostatic forces
- ◊ Dynamic forces
- ◊ Gravity forces
- ◊ Friction forces on saddles and expansions

Anchors are generally not required for buried pipe except at horizontal bends with large deflection angles, and overbends with high uplift forces for which the backfill alone cannot provide enough resistance.

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

In countries with 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 by means of protection trenches, walls, wire mesh, etc.

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

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

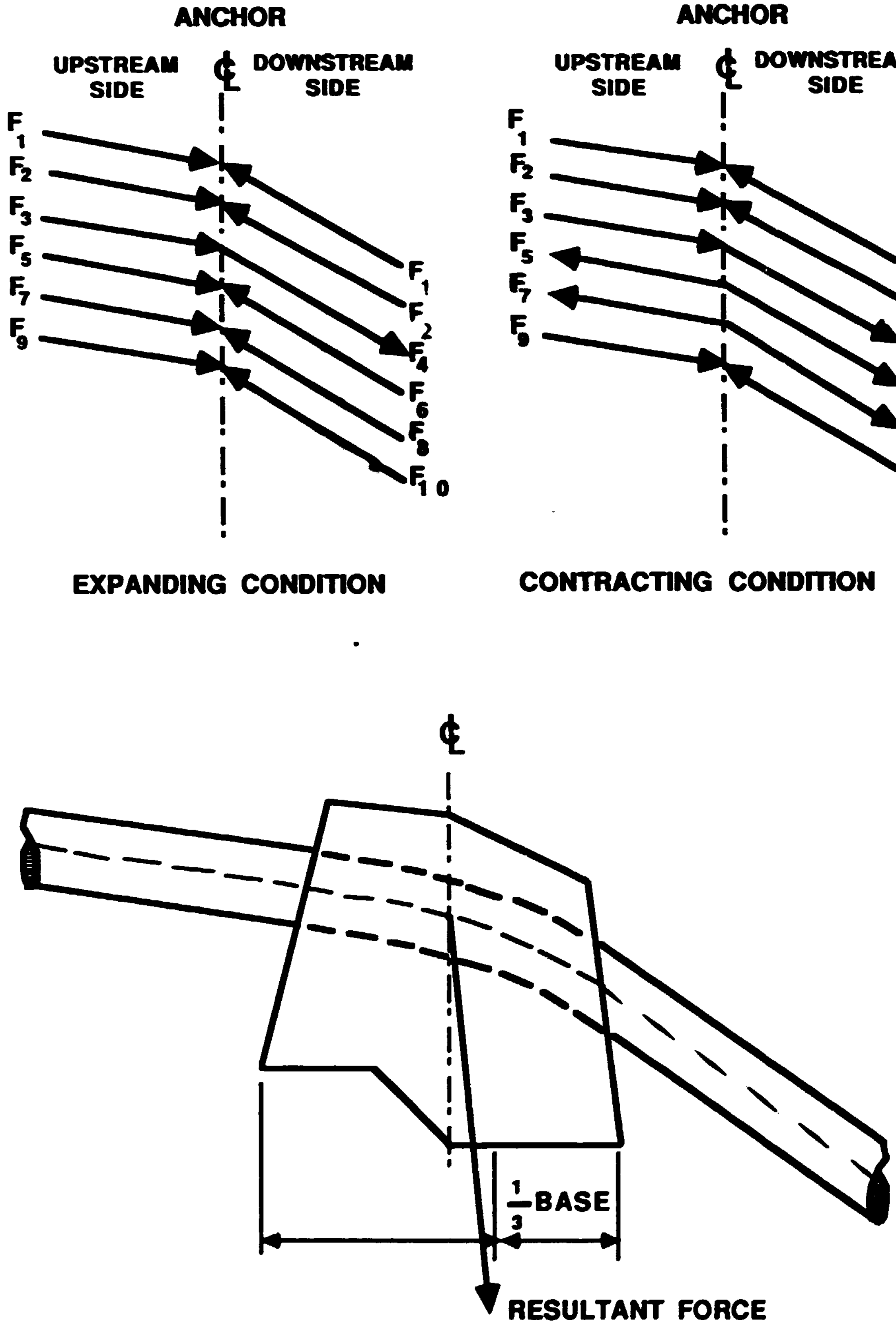


Fig. 10.9 Forces acting on an anchor block

The main forces acting on an anchor block are the following (see Fig. 10.9):

10.14.1 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 of the pipe in m^2
H =	Maximum water head in m

10.14.2. The dynamic forces acting against the outside bend:

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

γ =	weight of water = 1000 kg/ m3
Q =	Flow of water in m3/s
v =	velocity of water in m/s
g =	acceleration of gravity = 9.81 m/s2

10.14.3. Downhill force due tc dead weight of the pipe, from anchor uphill to expansion joint.

$$F_3 = P_1 \sin x$$

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

10.14.4. Downhill force due to dead weight of pipe from anchor downhill to expansion joint.

$$F_4 = P_2 \sin y$$

where:

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

10.14.5. Sliding friction of pipe on blocks due to expansion or contraction uphill from anchor:

$$F_5 = f_5 \cos x \left(P_1 + W_1 - \frac{p_1}{2}\right)$$

the coefficient f_5 of friction of pipe on blocks 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 in kg
W_1 = Weight of water in pipe P_1 in kg
p_1 = weight of pipe including water from anchor to adjacent uphill block in kg

10.14.6. Sliding friction of pipe on blocks due to expansion or contraction downhill from anchor:

$$F_6 = f_6 \cos\ y \ \left(P_2 + W_2 - \frac{p_2}{2}\right)$$

f_6 = coefficient of friction of pipe on blocks $= f_5$
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 block in kg.

10.14.7. Sliding friction of uphill expansion joint:

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

where:

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

10.14.8. Sliding friction of downhill expansion joint:

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

where: $f_8 = f_7$

10.14.9. 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/m^3
D = Pipe diameter in m
H = Maximum water head in m
t_9 = wall thickness of pipe wall in m

10.14.10. 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/m^3
D = Pipe diameter in m
H = Maximum water head in m
t_{10} = wall thickness of pipe wall in 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 (See Fig. 10.9).

In countries with seismic activity the corresponding earthquake forces must be added.

Forces due to expansions or contractions of the pipe have not been considered in the above-mentioned system.

In figure 10.10 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 Figs 10.10 and 10.11 schematic support of pipes joined with Dresser couplings are depicted. A Ring girder type support and a 120° saddle type support are shown on Fig 10.11, an application for steep hill sides or very rough terrain.

In Fig 10.11 intermediate supports for small diameter pipes are shown, center type supports, and "two and one" type support to be use with the combination of Dresser coupling.

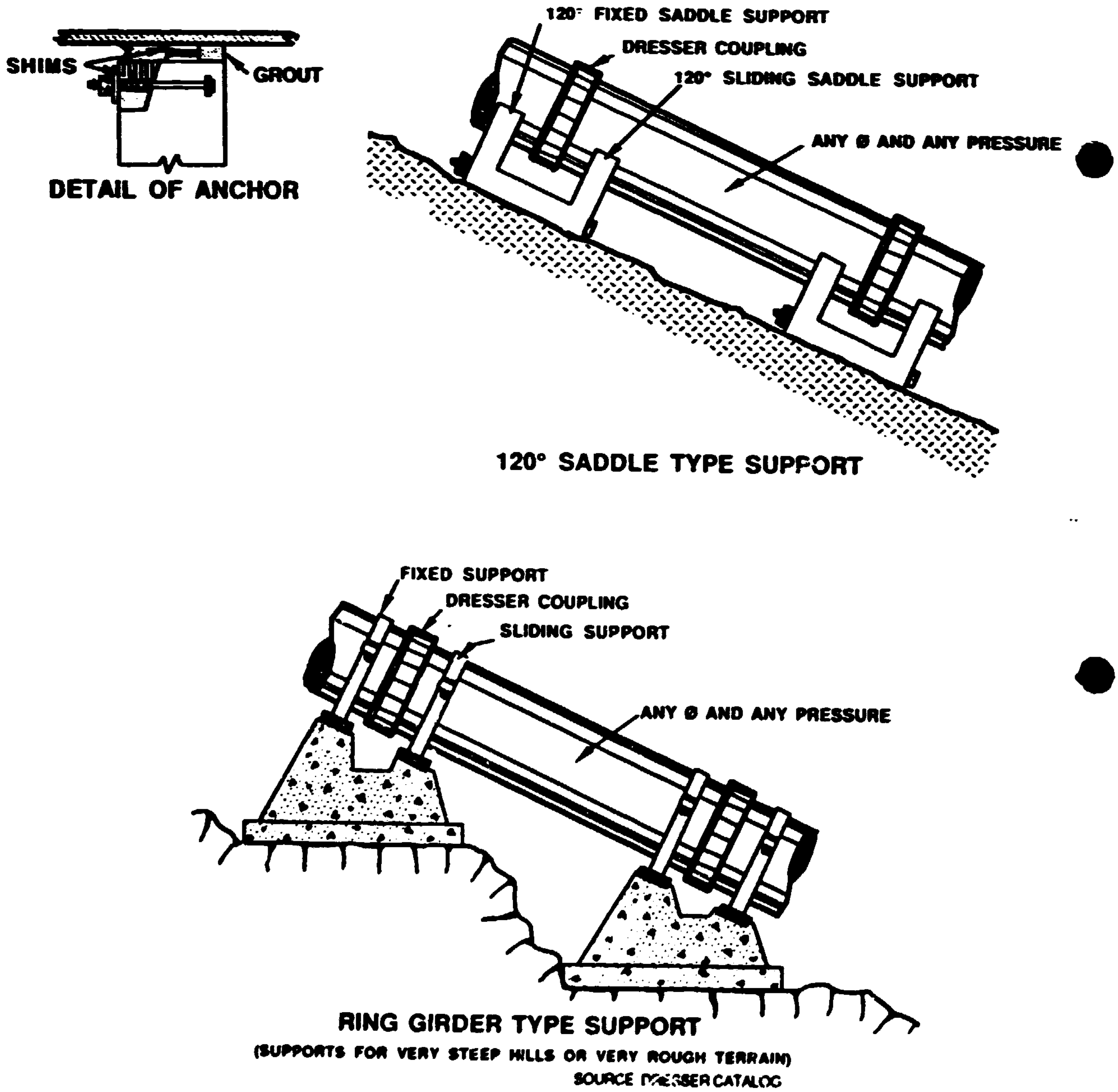
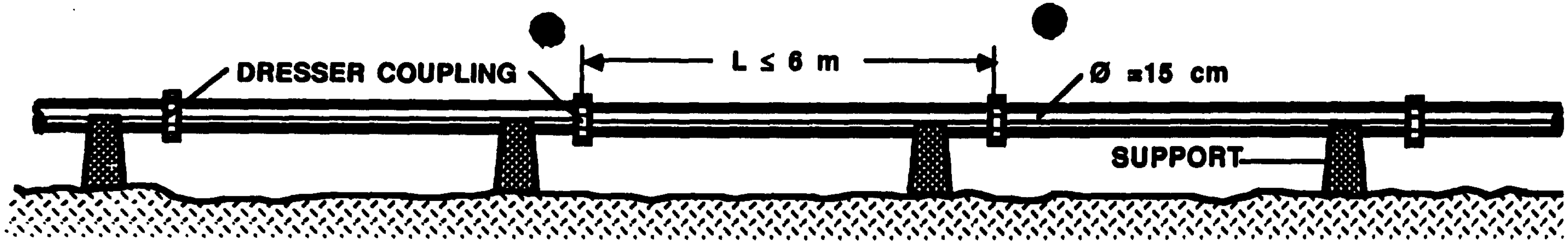
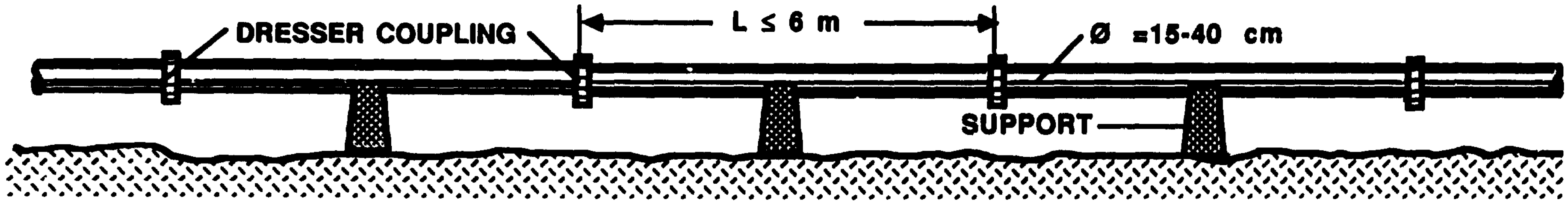


Fig. 10.10 Support of pipes joined with Dresser coupling



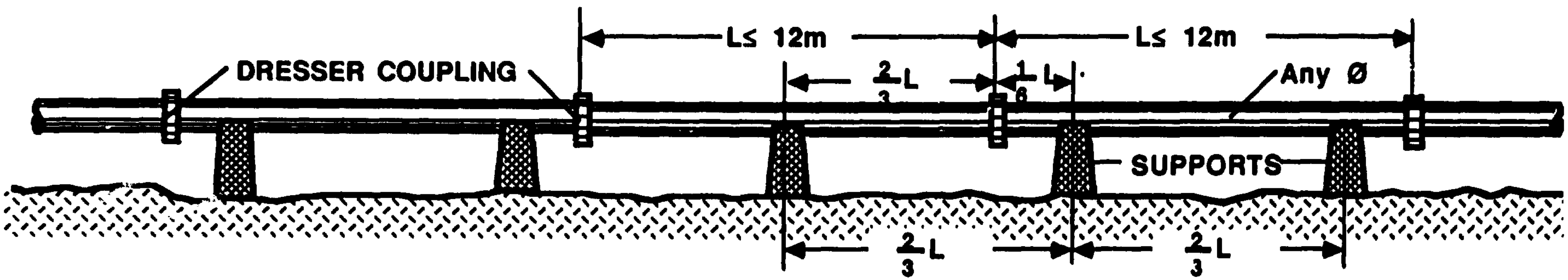
SUPPORT FOR SMALL DIAMETER ONLY

SUITABLE FOR ANY PRESSURE PROVIDED PIPE IS WELL ANCHORED TO SUPPORT THE PRESSURES
(CONSIDER WHERE REQUIRED THE EFFECT OF FROST DURING THE WINTER TIME)



CENTER TYPE SUPPORT

SUITABLE FOR PRESSURES UP TO 12 kg/cm2 MAXIMUM. PIPE MUST BE WELL ANCHORED TO SUPPORTS



"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 MINOR SLOPE
EACH LENGTH OF PIPE MUST BE ANCHORED TO ONE (AND ONLY ONE) SUPPORT

FIG. 10.11 Some types of intermediate supports

10.15 Corrosion Protection

10.15.1 Generalities

The anticorrosive protection is an important factor to assure the durability of the penstock. Two aspects must be considered i.e. "Rust removal" and "Surface treatment and painting".

Both treatments must be done in a climatized room in a suitable work shop, according to the recommendations of the pipe supplier. The application of protective media will be done inside and outside the pipes, whereby the following aspects will be taken into consideration:

• Each end of the pipes shall be freed from painting to a length of about 200 mm; both ends will then be immediately protected against damage during transport and storage prior to final installation. The pipes will be welded on the job site; the paint free zone ensures that perfect welding of the pipes joints is obtained. Without the above precaution, the painting and corrosion protection would be permanently damaged whilst welding the joints.

• Finishing on site will be made after termination of the welding on the job site (a process that is possible only for pipes with a diameter of 600 mm or more); for diameters of less than 600 mm the use of couplings is recommended so that the pipe can be properly protected inside and outside against corrosion.

In case of a welded pipe, the finishing work will consist mainly of:

- Careful brushing and cleaning:

All surfaces to be painted shall be thoroughly cleaned of all dirt, dust, grease, cement scale or oil before the application of paint. Oil and grease shall be removed with solvents not harmful to the surface. If required by the paint system, the surface shall be finally cleaned with water.

- Drying just before the application of the paint:

Surfaces shall be dry unless dampening is required for a particular finish material. Any surface contaminated during paint application shall be cleaned to the degree specified before painting is continued.

- Application of three layers of EPOXY TAR with a total thickness of not less than 0.5 mm

In colder regions, where frost may occur, or in areas where rockfall is expected it may be advantageous to bury the pipe and protecting it with 0.6-1.0 m of fill. However, sight control of a buried pipe becomes impossible; its maintenance becomes very difficult and the corrosion risks increases. The magnitude of excavation and the volume of fill required increase considerably the cost of installation.

The buried solution should therefore be adopted only when overriding reasons such as military, touristic or environmental considerations makes it necessary.

Even for environmental reasons it is doubtful whether the advantages of the buried penstock outweigh its disadvantages: whilst excavating, materials deposited along the penstock trench, often impair the growth of surrounding vegetation. A better solution is to paint the conduit with an appropriate colour and to plant trees on both sides of the conduit.

10.15.2 Rust removal

Heavy rust on steel surfaces shall be removed by chipping before processing. The following codes and specifications shall be valid:

SIS 055900:	Swedish Corrosion Institute
SSPC:	Steel Structures Painting Council
DIN 55928:	German Industrial Standard
BS 4232:	British Standard
or equivalent.	

The required grade of surface preparation for painting to be applied is specified and described in the Swedish Standard 055900. The grade of actual preparation shall be compared with the photo index given in the above standards.

Whenever possible mechanical removal (sandblasting, shot blasting and/or wire brushing) of rust and mill scale shall be made at the manufacturers' works.

10.15.3 Surface treatment and painting

De-rusting by sandblasting should be made near the temperature dew point (not less than 3° C above it); otherwise no sandblasting shall be performed. Sandblasted steel parts shall not be touched with bare hands.

Forging scale and foreign matter shall be removed so that whatever remains appears only as shiny discolourations. The surface shall then be cleaned either with a vacuum cleaner, dry and oil-free compressed air or with a clean brush. After cleaning, the surface appearance shall correspond to the photo index for SA 2 1/2 (Grade Sa 2 1/2 according to the Swedish norm SIS 055900 or equivalent like the SSPC - SP 10, DIN 55928 - Sa 2 1/2 BS 4232 - Second Quality).

Sandblasting should be made until the metal surface is absolutely clean. All forging scale, rust and foreign matter shall be totally removed. Dust cleaning as described above for Sa 2 1/2.

After cleaning the surface shall be purely metallic and shall correspond to the photo index for Sa 3 (Grade Sa3 according to Swedish norms SIS 055900 or equivalent like SSPC - SP 5, DIN 55928 - Sa 3, BS 4243 - First Quality).

The painting system to be used depends on the weather conditions prevailing. The following Table 10.7 gives approximate indications in relation to paint and corrosion protection systems.

TABLA 10.7 Paint System and Application Method to be used for Penstocks of Small Hydropower Plants

EXTERNAL CONDITIONS	SURFACE PREPARATION	PAINT SYSTEM	NOMINAL DRY FILM THICKNESS (in mm)	APPLICATION METHOD			
				BRUSH	ROLLER	SPRAYING	AIR-LESS
Buried penstock conduits with temperatures up to 100 °C	Sa 2 1/2 - 3	**Prime Coat:** 1 x zinc rich primer (2 component)					
		Base: Epoxy resin	1 x 0.050	X	X	X	X
		Base: Epoxy resin	1 x 0.080	X	X	X	X
		Finish Coat: 1 x tarepoxy resin (2 component)	3 x 0.150	X	X	X	X
Above ground penstock with temperatures up to 100° C in dry or moist salty water	Sa 2 1/2 - 3	**Prime Coat:** 1 x zinc rich primer (2 component)					
		Base: Epoxy resin	1 x 0.050	X	X	X	X
		Intermediate Coat: 1 x micaceous iron oxide paint (2 component)					
		Base: Epoxy resin	1 x 0.080	X	X	X	X
		Finish Coat: 2 x enamel, micaceous iron oxide, paint, coloured (2 component)					
		Base: Epoxy resin or 2 x enamel paint (2 component)	2 x 0.080	X	X	X	X
		Base: Polyurethane resin	2 x 0.060	X	X	X	X

CHAPTER 11

11. SURGE TANK

11.1. Generalities

Surge tanks, are devices that control hydraulic transients in hydraulic systems; they are usually open at top and connected to the penstock pipe at bottom.

Surge tanks are less often used in relation to small hydropower schemes owing to economical (cost!) and technical reasons. In many cases hydraulic surges can namely be taken care of by over-dimensioning forebay and spillway accordingly. This is the case of practically all hydro schemes with heads below 100 m where the surge problems are solved either by the adduction of a free flow channel or conduit or an enlarged forebay which serves at the same time as general or operative reservoir and overflow. However, whenever a surge tank is required to meet specific conditions, the following remarks will be found useful:

Single surge tanks will be used as a rule; they should present little resistance to the flow of water in and out of the penstock; they should also be built as economically as possible

11.2 Pressure adduction conduits

A surge tank is more likely to be required when the adduction conduit or tunnel is under pressure. Most frequent cases are also encountered for installations in steep slopes and/or complicated topographical conditions, such as rock edges, clifts, possible forests, etc., and for which heads are higher than 150 m.

Fig. 11.1 illustrates the case of a junction penstock / pressure conduit (Point C) where a normal forebay is no longer convenient or possible. Effectively the adduction system composed of a pressure conduit of length L' and a penstock of length L, forms a long uninterrupted pressure conduit. Such a long conection may become dangerous on the occurrence of a water hammer; it is therefore advisable to replace an enlarged forebay with a surge chamber (See Fig. 11.1) with a vertical shaft of relatively large volume and sufficient height as shown on the above figure, the conection to the pressure conduit is made at the bottom N of the chamber. With this arrangement the continuity of the pressure system is separated in two lengths L and L'.

Fig. 11.1 shows the surge tank in its simplest form. As long as the plant is out of service, the water level in the surge tank is at the static level (Point P) equal to the level of the intake.

Surge shafts should be designed in such a manner as flow-over does not occur, even during a transient phenomena; in other words its height should be adequate. Some isolated cases are known where overtopping is allowed (See remarks further on).

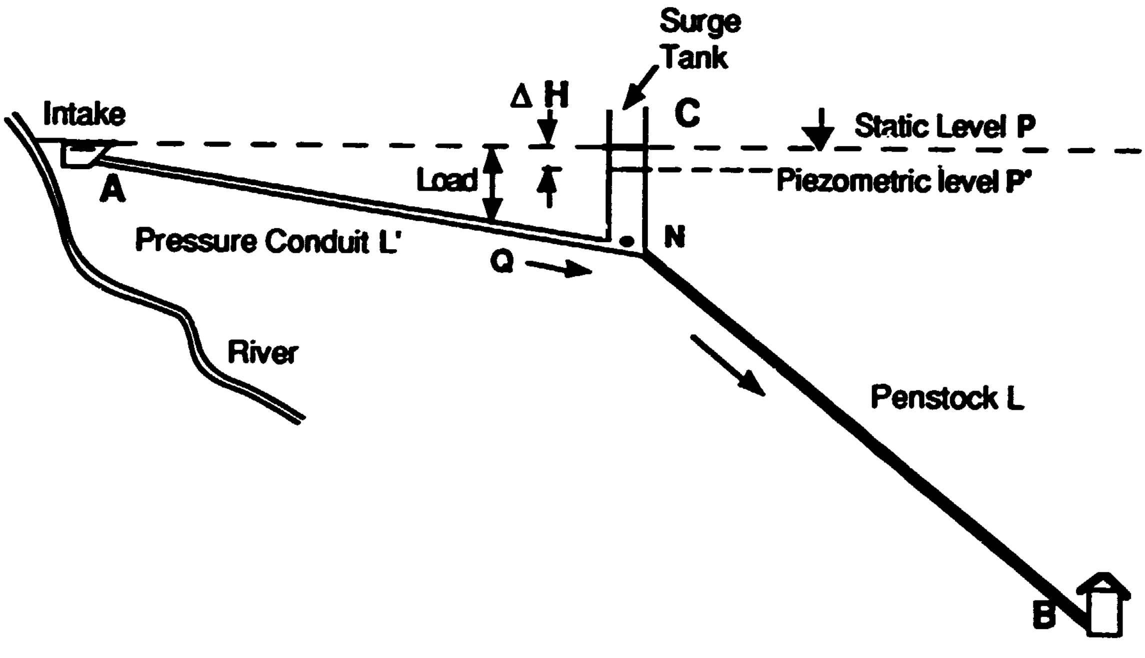


Fig. 11.1 Description of system with surge tank

As soon as the plant is put into operation, with water flow Q, the water level P of the surge chamber, will sink and stabilize at level P', its piezometer level; the pressure differential Δh corresponds to the friction loss in the pressure conduit with flow Q.

A water hammer will occur if, during operation the flow Q is rapidly throttled. The resulting pressure wave in the penstock will spread into the surge tank. Without surge tank the water hammer would proceed along the pressure conduit until the intake; with a surge tank present, only the penstock will be affected by the pressure wave.

As explained previously a water mass oscillation will occur in the penstock and surge shaft. This mass oscillation may last several hours, whereas the pressure waves (upwards and downwards) will usually be damped within a some seconds.

For this purpose it is necessary to examine the worst condition i.e. total and instantaneously closing of the butterfly valve at the bottom of the penstock, cutting the flow to the turbines. Rapid immobilization of the water in the penstock will produce a water hammer. This will not occur instantaneously in the pressure conduit L' where Q flowed before the interruption, however the water level will then rise in the surge shaft to a level higher than the normal static level; it will reach its maximum at the same time as the water flow stops in the pressure conduit. Physically that means that the kinetic energy of the flowing water mass in the pressure conduit has been utilized firstly to lift the water in the surge shaft to its maximum and secondly to overcome flowing friction.

After reaching its maximum level in the surge shaft the flow of water in the pressure conduit will be reversed, whereby the lowest water level reached will be lower than static level. A water flow will be established in the opposite direction in the pressure conduit, utilizing the potential energy stored in the shaft between maximum and minimum levels.

This processes will be repeated to produce water oscillations which eventually will be damped by the internal friction.

The other limit condition occurs when the plant, after shut down, is re-commissioned. A water flow Q will therefore appear, creating an underpressure in the pressure conduit L' from the intake to the surge tank. At first the required flow Q cannot be delivered by the pressure conduit L because at its both ends the water is at the same level (static level). It is therefore clear that it will be the surge shaft which will first supply the needed flow Q; however its level will sink, creating a level difference between intake and shaft, or in other words a piezometrical level difference in the pressure conduit L'; water will progressively move downwards the conduit, its level in the shaft being lower than the dynamic pressure which will prevail once the system flow has stabilized. Until stability has been obtained an oscillation phenomena will occur which be progressively damped.

11.3. Dimensioning of Surge Tank

As has been explained above, surge tanks take care of transient mass movements of the water during transient events of the power plant i.e. shut-down, commissioning and load variations.

The surge tank protects the penstock from potential damages to its structure. It is relatively unusual to design a surge tank with overtopping, although some cases are known where surge chamber spillways have been used.

Foremost consideration in relation to deciding whether a surge tank is required or not is to consider the relation penstock length to gross head.

If

$$\frac{L}{H} \geq 6 \qquad (11.1)$$

a surge chamber will usually be required for small hydro power plants (A relation of $L/H \geq 4$ is applicable to medium or large plants).

The two main criteria a) and b) to guarantee the stability of the system are the criteria of Thoma, that can be expressed in the following form:

a) $$h_c \leq \frac{H_n}{2} \qquad (11.2)$$

b) $$A_s = n_1 A_{Th} \qquad (11.3)$$

Where: $$1.5 < n_1 < 2.0 \qquad (11.4)\Delta$$

and $$A_{Th} = \frac{V^2}{2g} \frac{A\,L}{H_n h_c} \qquad (11.5)$$

or $$1.5 \frac{V^2}{2\,g} \frac{A\,L}{H_n\,h_c} \leq A_s \leq 2.0 \frac{V^2}{2\,g} \frac{A\,L}{H_n\,h_c} \qquad (11.6)$$

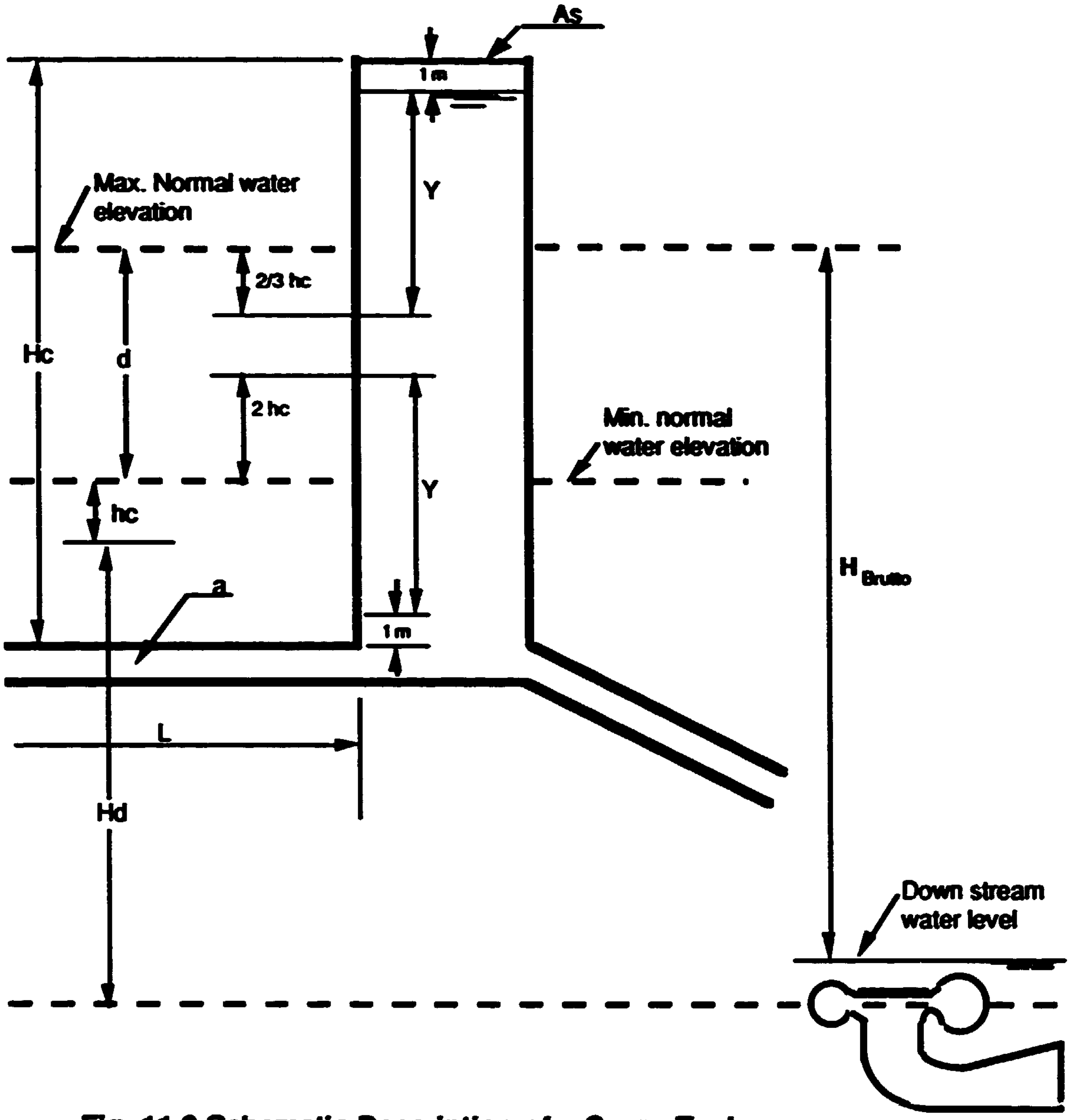


Fig. 11.2 Schematic Description of a Surge Tank

Where:

	A =	Area of the penstock section, in m^2
	A_s =	Area of the vertical shaft in m^2
	A_{Th} =	Area of the vertical shaft after Thoma
	d =	Maximum depletion of the reservoir, in m
	g =	Acceleration of gravity, (9.81 m/s^2) 1.5 and 2.0 are the values of the safety coefficients set by Thoma.
	h_c =	Total head losses in the penstock, in m
	H_c =	Height of the surge chamber, in m
	H_n =	Net head (minimum static head less h_c), in m
	L =	Length of the penstock pipe, in m
	V =	Water velocity in the penstock, in m/s
	Y =	Oscillation of the water in the surge chamber, in m

The total head losses h_c are the losses at the entrance, in bends and in valves, as described in chapter 6, plus the friction losses. The friction losses can be calculated using the Strickler formula:

$$h_t = L\left(\frac{V}{k\ R^{2/3}}\right)^2 \qquad (11.7)$$

Using k = 80 for concrete, and k= 100 for steel

and $$h_c = K_1\frac{V^2}{2\ g} + K_2\frac{V^2}{2\ g} + K_3\frac{V^2}{2\ g} + \text{etc.} \ldots + h_t + \left(\frac{V^2}{2\ g}\right) \qquad (11.8)$$

The coefficients K_1, K_2, K_3, etc, are as described in chapter 6

The value of the maximum oscillation Y in the surge chamber can be computed with the following formula:

$$Y = V\sqrt{\frac{L\ A}{g\ A_s}} \qquad (11.9)$$

The height of the surge chamber can be computed with the following formula:

$$H_c = d - \frac{8}{3}h_c + 2Y + 2 \qquad (11.10)$$

For better comprehension of the symbols, see Fig 11.2

CHAPTER 12

12. POWERHOUSE

The principal purpose of the powerhouse building is to protect the generating equipment. Therefore it has to be built in a place as safe as possible, i. e. it must be protected from landslides, from river floods and from 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 able to maintain at all times dry conditions of the ground below the foundation slab.

A permanent access to the powerhouse must be provided.

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 as 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, because 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 only has to be provided. The roof may also serve solely as protection against the solar radiation. Final decisions have to be made according to the local conditions.

Below the powerhouse, provisions should be made for the discharge of the turbinated water in a channel that communicates directly with the tailrace channel. There are small differences in 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 itself is a channel that can be treated in most cases as the channels mentioned in Chapter 8 of this report.

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EXAMPLES

EXAMPLE 1

CONVENTIONAL WATER INTAKE

(Without Desilting Basin)

$Q = 1\ m^3/s$

EXAMPLE OF A CONVENTIONAL WATER INTAKE FOR Q= 1 m3/s

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

This example refers to a water intake on sound rock foundations in a river bed. Normal transport of sediments which generally occurs during floods has been assumed .

A maximum design flood (Q_{Max}) of 12 m3/sec has been adopted, about equivalent to a 1000-year flood.

The intake and the sluice channel are located on the outside bend of the river to allow diversion of water with less sedimentation, and easy removal of the sediments through the sluice channel.

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

The weir, intake structure and spillway, as well as the sluice channel, have been designed in masonry, which for this type and size of construction is universally well accepted, is cheaper than concrete and has performed well in several similar plants elsewhere in the world.

The gate and trash rack will be built with light PC-300 reinforced concrete.

The sluice system consist of with the following main parts:

- ◊ An inlet sill (that favours the flow of sediments)
- ◊ A sluice channel with a gradient of S = 0.08 to ensure good hydraulic conditions and to avoid that sediments will block this important element.
- ◊ A sluice gate of 1.00 m width and 1.00 m height, for mechanical operation, either hand-operated or preferably motor-operated.
- ◊ With the gate open, the free-flow flushing capacity of the sluice channel is 3 m^3/sec, being considered appropriate and sufficient for this type of intake; however the local hydrological conditions should be checked in each case, and a higher capacity should be adopted if the flow is larger.

The trash rack should be inclined by about 15° to the vertical, in order to allow maintenance and easy cleaning.

The trash rack is 2.50 m wide and 0.80 m high, and the arrangement of the bars is as shown in Fig. Nr. EX1-1

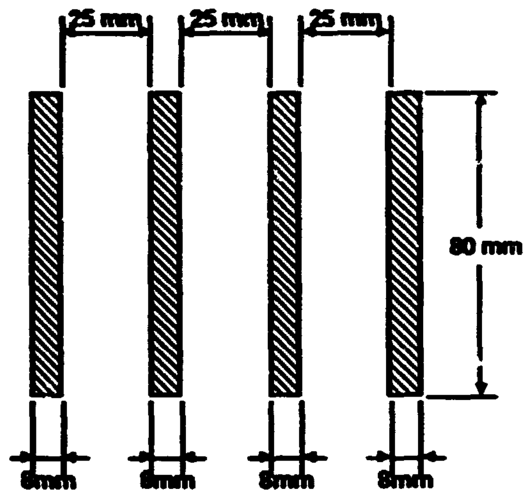


Fig. EX 1-1 Detail of trash rack steel bars

An approach velocity in front of the bars of 0.5 m/s has been assumed; this is a conservative value obtained from experience in other similar intakes (This velocity ensures also that a minimum of sediment will pass through the rack).

The arrangement of the sluice channel and the trash rack encourages sedimentation in front of the intake and ensures safe intake operation with an adequate safety margin.

The depth and volume of sediments have to be checked regularly in order to ensure periodical flushing of sediments through the sluice channel. The importance of regular flushing operations should be emphasized in the operation manual for the maintenance personnel.

The gate should be operated at regular intervals, even when no sediments are present, to ensure its satisfactory operation at any time. It should not remain out of operation for a long period of time to avoid blockage.

For inspection or repair of the flushing gate, a slot for placing stop logs, formed of horizontal elements, has been provided. The stop logs can be installed during low flow in the river.

The intake structure is provided with aeration and access openings.

At the intake entrance, stop log slots have alsobeen provided so that the approach canal can be emptied for inspection. It is important to mention that during inspection the sluice gate has to be kept open.

A control section of 0.60 m x 0.60 m allows any excessive inflow during floods to be reduced, see Drawing Nr.40093-1-2 numeral 13.

By means of a small sluice gate of 0.20 m x 0.30 m rest sediments passing into the canal, can be flushed see Drawing Nr. 40093-1-1, numeral 14.

The approach canal has an additional feature; a lateral spillway, with a width of 4.00 m, To evacuate any excess flow that could occur during floods, thus preventing the danger of a canal overflow.

The crest elevation is 199.70 m a.s.l. A sill and flood protection beam (see Drawing Nr.40093-1-2 numeral 16), will evacuate any excess flow, forcing the water excess to spill through the lateral spillway.

The bottom of the canal has been set at elevation 199.00 m a.s.l., with a slope of S = 0.0012, in order to provide satisfactory subcritical flow *(it is not recommended that the gradient of the sluice channel bottom be less than S = 0.001).*

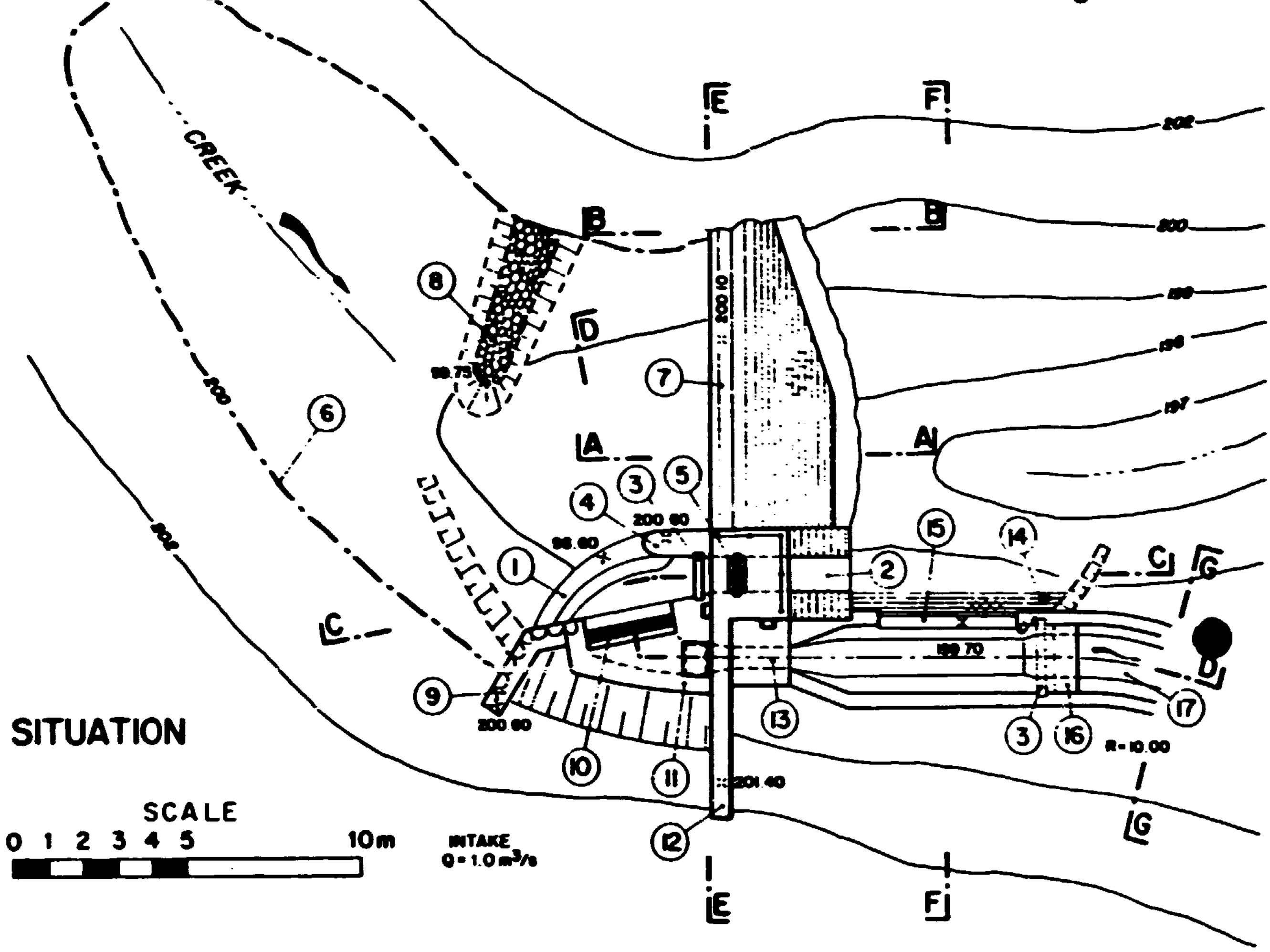
The bed of the canal for a normal design discharge of Q = 1 m^3/s is 1.35 m wide and made of PC-300 concrete.The inclined slopes are of masonry with a 5:1 inclination(horizontal to vertical). The maximum water depth should be 0.70 m.

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

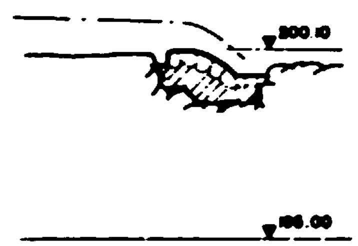
CONVENTIONAL WATER INTAKE FOR Q = 1 m^3/s

LEGEND

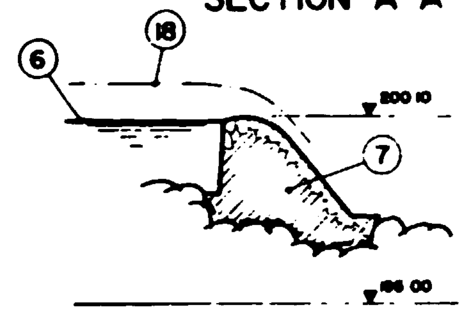
1. SLUICE SILL EL. 198.00 m a.s.l. IN MASONRY
2. SLUICE CHANNEL S = 0.08
3. STOP LOGS SLOTS
4. GUIDE WALL CREST EL. 200.60
5. SLUICE GATE 1.00 m x 1.00 m
6. NORMAL RETENTION EL 200.00 m a.s.l.
7. WEIR IN MASONRY CREST EL 200.10 m a.s.l.
8. MASONRY TRAINING WALL
9. INTAKE WING WALL IN MASONRY
10. TRASH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



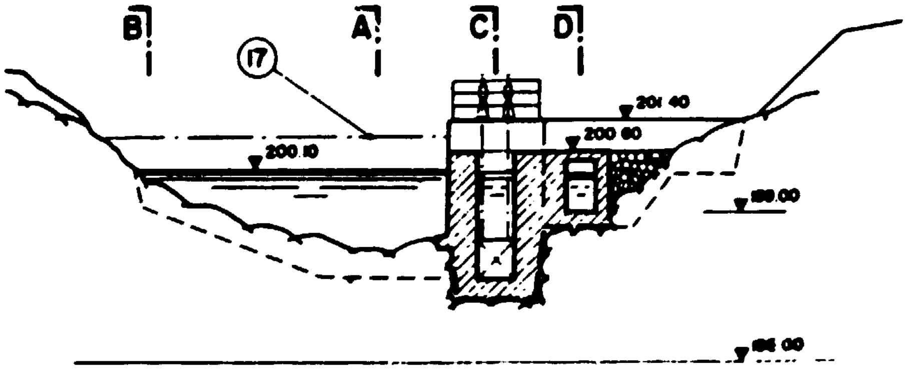
SECTION B-B



SECTION A-A

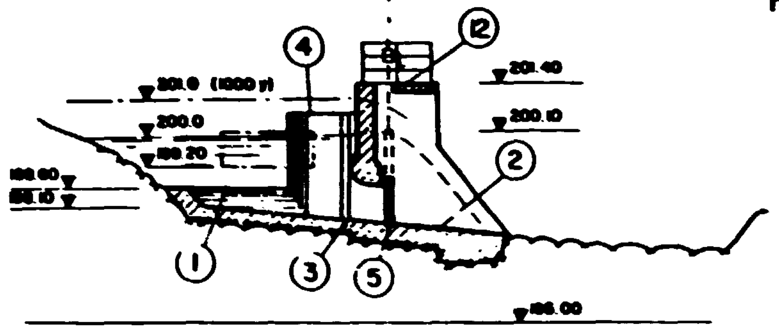


SECTION E-E



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION			EXAMPLE 1
Engineers + Consultants ZURICH - SWITZERLAND	DRA:	SGR	DRAWING Nr. 40093 - 1 - 1
	DESIGN:	BU	DECEMBER 1988
	APP:	KRU	

SECTION C-C



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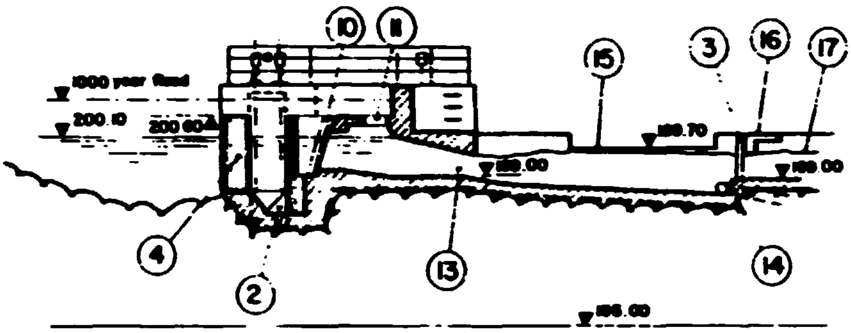
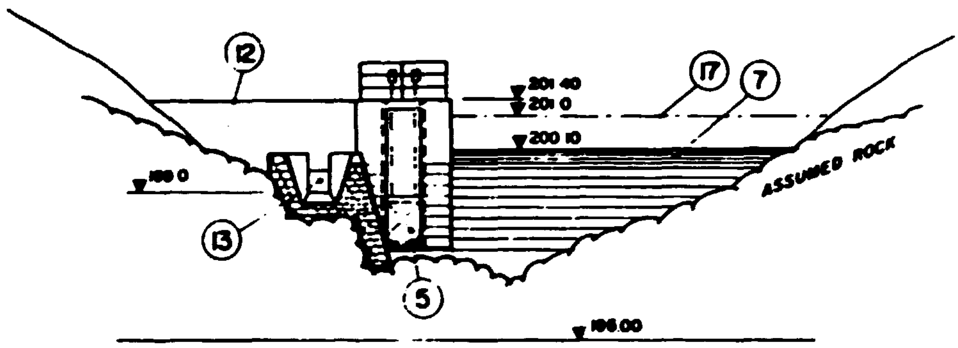
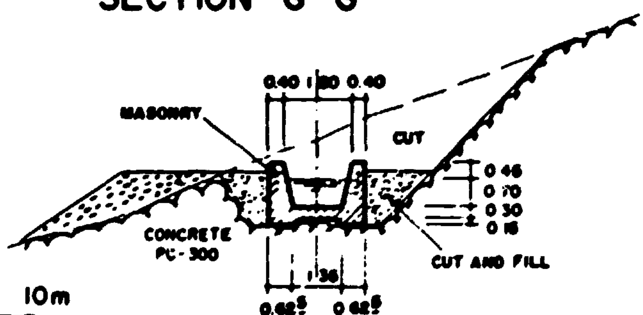


SECTION F-F



SECTION G-G



SCALE
0 1 2 3 4 5 10m

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 1
Engineers + Consultants ZURICH - SWITZERLAND	DRA: SGR DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 1 - 2 DECEMBER 1988

Engineers + Consultants

EXAMPLE 2

CONVENTIONAL WATER INTAKE

(with Desilting Basin)

$Q = 2.6\ m^3/s$

EXAMPLE OF A CONVENTIONAL WATER INTAKE FOR Q = 2.6 m3/s

An example of a conventional (run-of-river) intake for a discharge of 2.6 m3/s is shown on Drawing Nr.40093-2-1 / 2-4

The foundations of this water intake is in a river bed at the end of a section of small gradient and gentle slope. The ground is gravel and coarse sand. The intake represents an appropriate solution for a wide valley, part of which has to be closed with a long, side dam.

It has been assumed that there is no heavy transport of bed load and that only some fine suspended sediments are carried by the river.

Intake, weir and dam These three structures have been designed to be generally made of PC-300 reinforced concrete.

The weir has been designed to safety pass a 1000-year flood, which corresponds to a discharge of the order of 22 m3/s, a normal average for similar projects of this type and size.

A small stilling basin is incorporated downstream of the weir, with its invert is protected with geotextiles and gabions.

A retaining wall with a face of 4 : 1 (vertical to horizontal) is located on the right side (a favourable slope for an earthfill dam!).

Ground samples taken from pits and trenches, served to ascertain the soil properties valid for the dam design, and to determine the most suitable design criteria.

Design criteria for small dams are beyond the scope of this report, However, the authors refer readers interested in this aspect the standard work published by the Bureau of Reclamation of The United States of America, entitled "Design of Small Dams".

The sluice channel has been dimensioned to pass a discharge of 1 - 4 m3/s with free flow and supercritical, conditions, it must have a slope of S = 0.08, which slope will also ensure that it will remain free of sediment deposits and operate at all times.

The selected design of the sluice channel, also ensures that the area in front of the intake entrance is kept clear. Each time the sluice gate is opened sediments in front of the trash rack can be flushed through the sluice channel, always provided the discharge is sufficient.

The sluice gate (see Drawing Nr.40093-2-1 and 2-3, numeral 10), has to be operated regularly in order to ensure it is always operative, thus avoiding undesirable surprises in emergencies. The installation of stop-log groove upstream of the sluice gate is recommended; it ensures that repairs to the gate will be easy to carry out.

The intake sill is set high enough to provide complete protection against gravel being washed into the intake canal.

The trash rack inclined at 75°, is made of steel bars, of 8 mm width and 80 mm height, with openings of 25 mm between bars.

The approach velocity to the trash rack should not exceed 0.60 m/s; the flow velocity through the rack is 0.80 m/s.

The intake gate (see Drawing Nr.40093-2-1 and 2-3, numeral 16) is 1.00 m wide and 1.00 m high. The relatively small section selected is necessary to restrict the flow in case of floods.

Downstream from the entrance, stilling racks reduce the flow velocity into the desilting basin, the later being situated immediately after the entrance canal.

The desilting basin for a conventional intake of 2.60 m3/s, is designed to retain all granular material of 0.20-0.40 mm diameter which has managed to pass the trash rack. The apron water velocity is 0.25 m/s, which means that the basin retains all particles larger than 0.20 mm.

Effectively particles larger than 0.20 mm are harmful to the equipment, and especially to turbines of the Francis or Pelton type. Crossflow turbines are less sensitive.

The main dimensions of a desilting plant designed for a discharge design of Q = 2.6 m3/s are the following (see Table 7.1 too):

Q = 2.60 m3/s
B = 3.50 m
L = 28.00 m
D = 3.10 m
W = 1.10 m

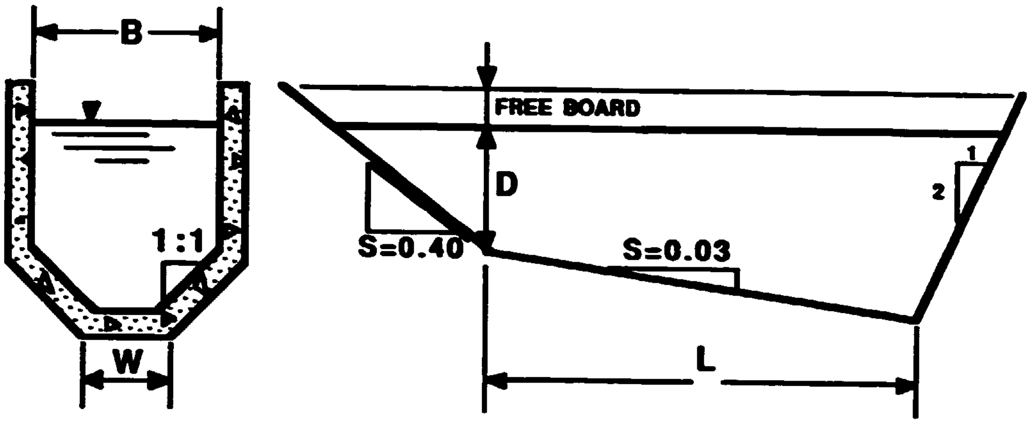


Fig. Ex 2-1 Main dimensions of the Desilting Plant for Q = 2.60 m3/s

The main dimensions of an adduction canal as taken from Table Nr. 8.3, are the following:

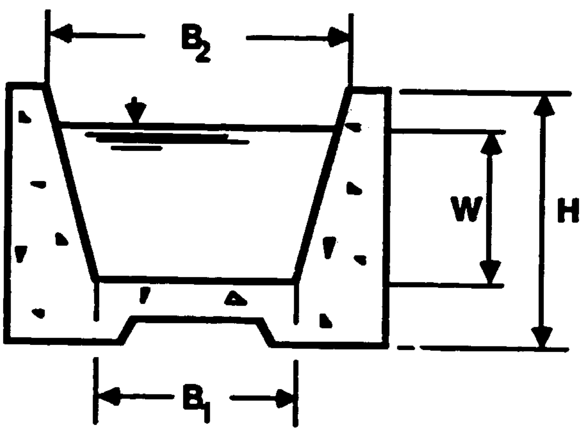


Fig. Ex.2-2 Cross section of canal example for $Q = 2.60\ m^3/s$

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

It is recommended that the radii of the curves shall not be less than 10-12 m , to avoid unnecessary spilling of water, and/or to avoid complicated curved sections with variable lateral gradients.

The slope of the canal should be about 0.0012, with a concrete bottom (PC - 300), lightly reinforced with steel bars; the walls of the canal can be made of masonry.

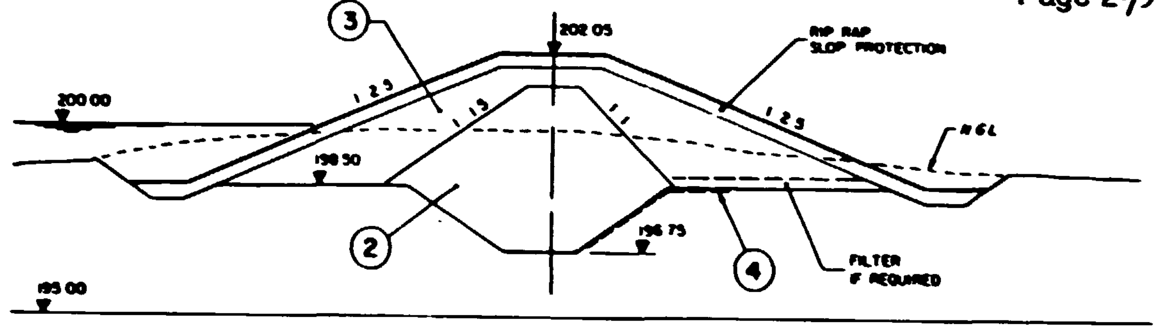
CONVENTIONAL WATER INTAKE for Q = 2.6 m³/s

LEGEND

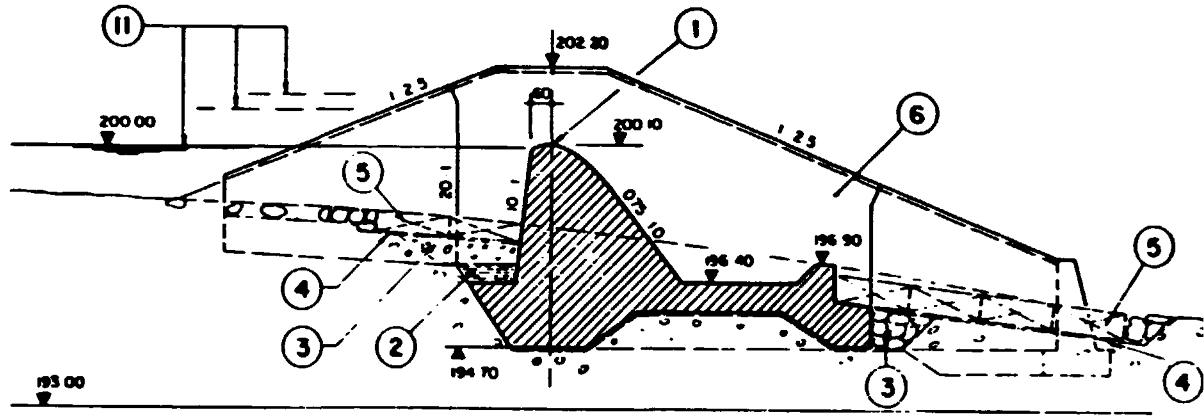
1. WEIR CREST EL. 200.10 m a.s.l.
2. COMPACTED CORE MATERIAL
3. SELECTED FILL COMPACTED
4. GEOTEXTILE
5. GABIONS 2.00 m x 1.00 m x 0.75 m
6. LEFT RETAINING WALL
7. SLUICE SILL EL. 198.45
8. SLUICE CHANNEL S = 0.08
9. GROOVE FOR STOP LOGS
10. SLUICE GATE 1.60 m x 1.60 m
11. RETENTION: NORMAL LEVEL 2000.00 m a.s.l. 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 OPERATING PLATFORM
16. ENTRANCE GATE 1.00 m x 1.00 m
17. HANDRAILS
18. STAIRS
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 m3/s
27. EXPANSION JOINT WITH WATERSTOP
28. SUPPORTING SLAB
29. TRANQUILIZATION RACKS.

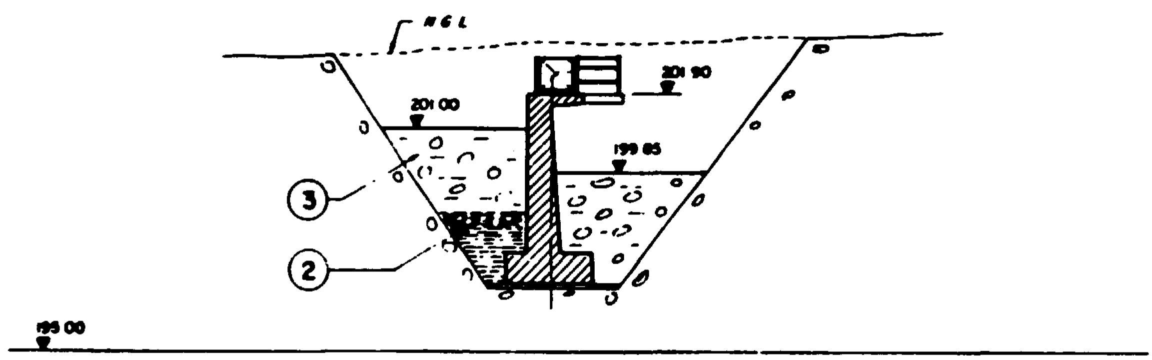
SECTION A - A



SECTION B - B

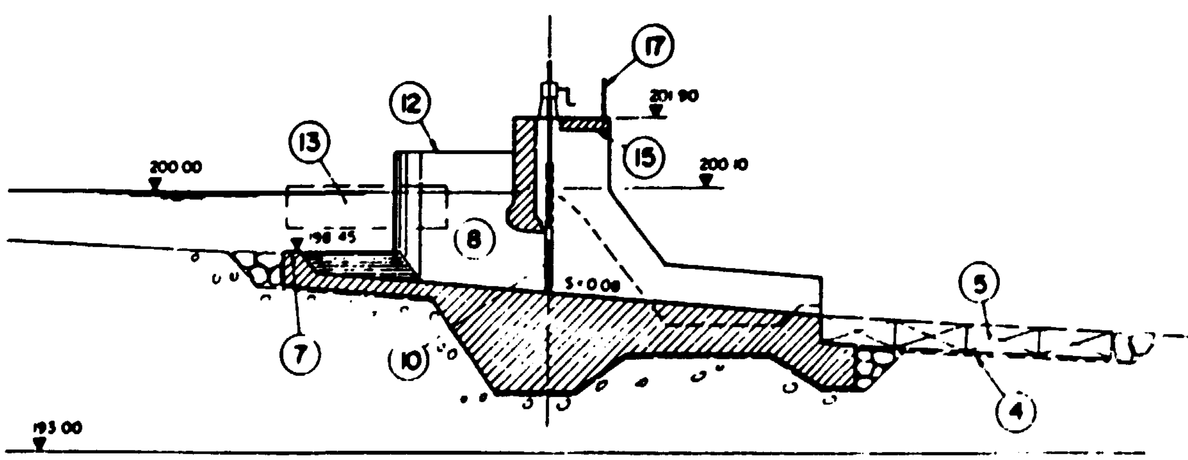


SECTION E - E

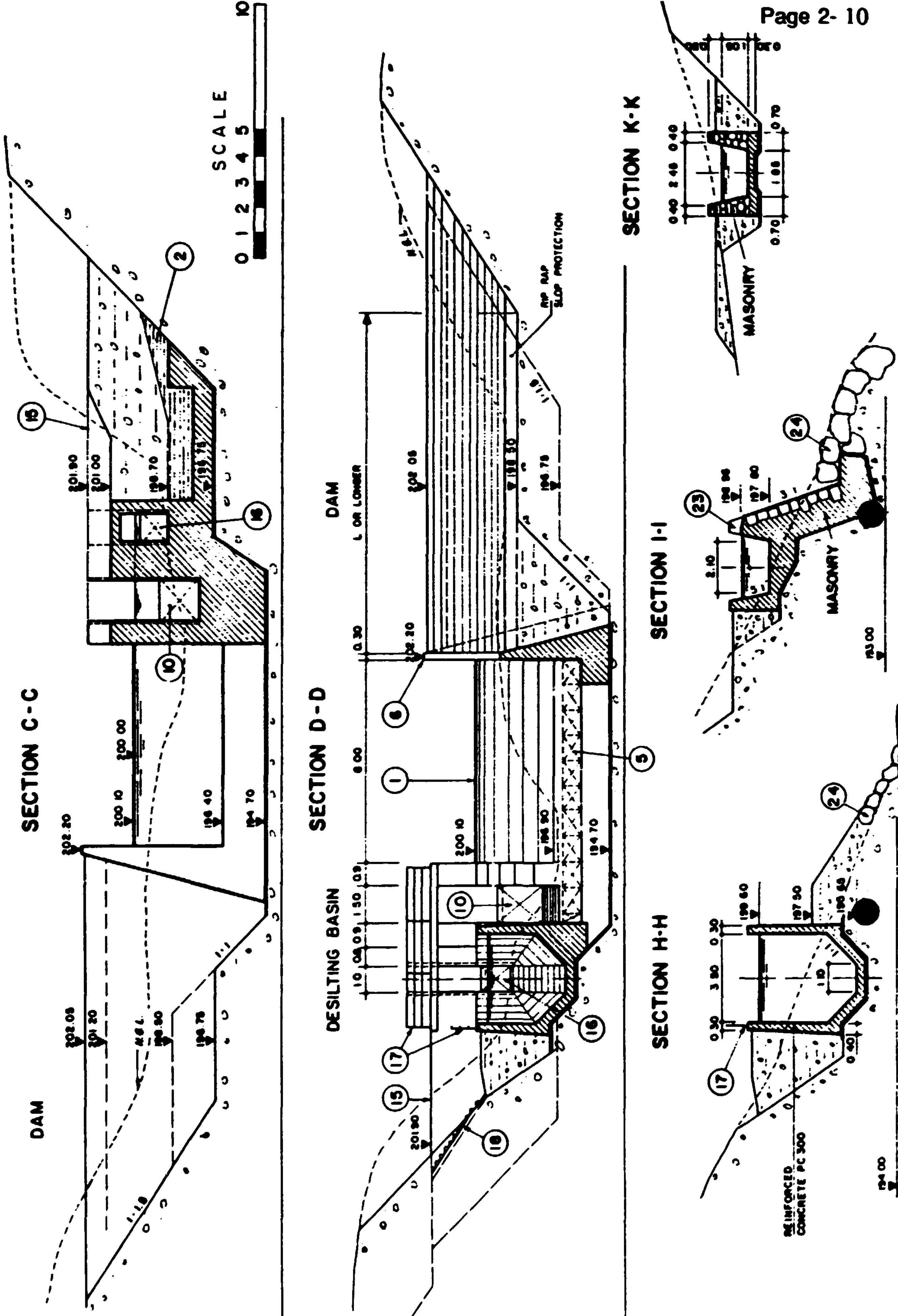


SCALE
0 1 2 3 4 5 10 m

SECTION L - L

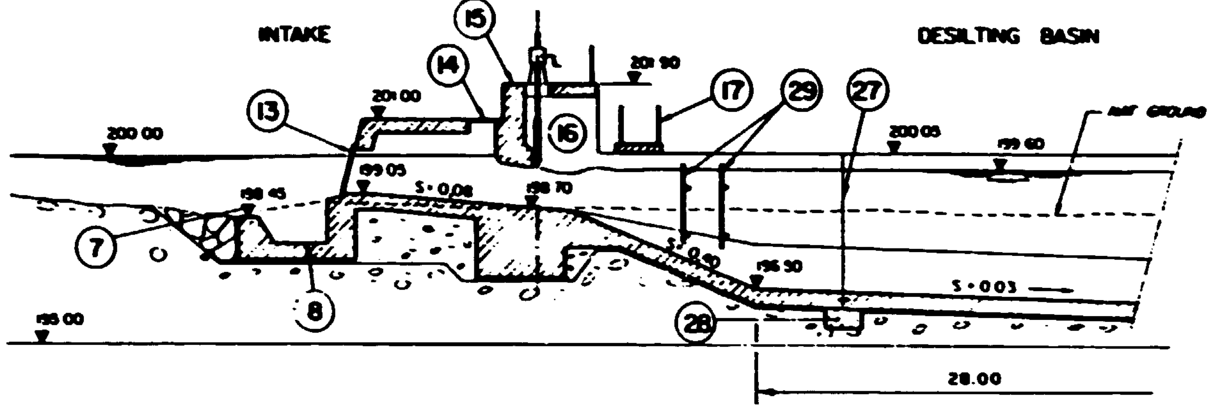


UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 2
Engineers + Consultants ZURICH - SWITZERLAND	DRA: ZUA DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 2 - 2 DECEMBER 1988

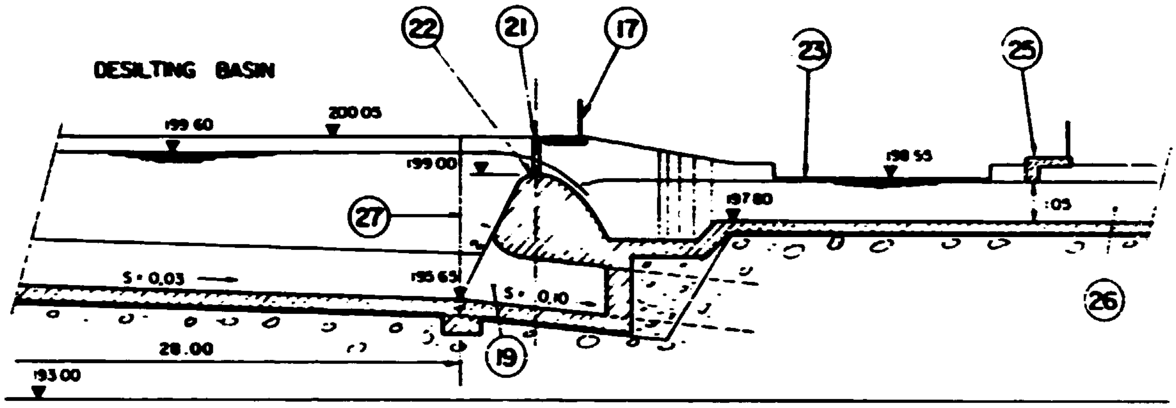


UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 2
Engineers + Consultants ZURICH - SWITZERLAND	DRA: MOL DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 2 - 3 DECEMBER 1988

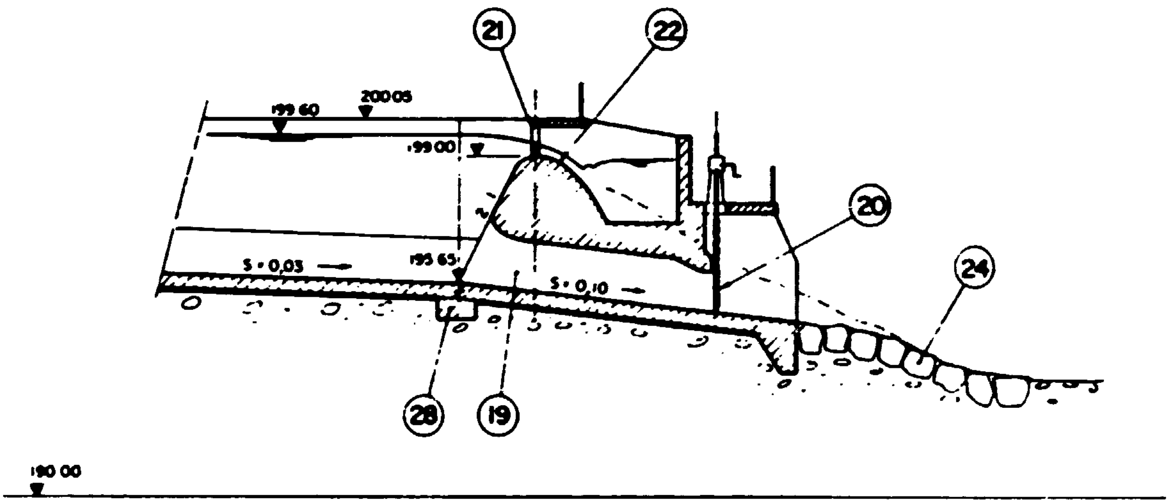
SECTION F - F
(UP-STREAM)



SECTION F - F
(DOWN-STREAM)



SECTION G - G



SCALE
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UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 2
Engineers + Consultants ZURICH - SWITZERLAND	DRA: ZUA DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 2 - 4 DECEMBER 1988

Engineers + Consultants

EXAMPLE 3

CONVENTIONAL WATER INTAKE

With Desilting Basin

$Q = 3.5\ m^3/s$

EXAMPLE OF A CONVENTIONAL WATER INTAKE FOR Q = 3.5 m3/s

An example of a conventional Water Intake (run-off-river type) for Q = 3.5 m^3/s is presented in Drawing Nr. 40093-3-1 / 3-3

The foundation of the structures has been assumed to be a favourable river bed on sound rock.

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

Q (10-year) = 18 m3/s

Q (50-year) = 24 m3/s

Q (100-year) = 30 m3/s

Q (1000-year) = 40 m3/s

Three cases have been considered:

Case 1. For a 100-year flood of (Q_{design}) 30 m3/s

◊ With main gate open — Q_{gate} = 22 m3/s
◊ Overflow over weir crest — Q_{weir} = 8 m3/s
◊ Approx. upstream water level — 200.85 m a.s.l.

Case 2. For a 1000-year flood of 40 m3/s

◊ With main gate open — Q_{gate} = 27 m3/s
◊ Overflow over weir crest — Q_{weir} = 13 m3/s
◊ Approx. upstream water level — 201.20 m a.s.l.

Case 3. Emergency case for a 1000-year flood

◊ Main gate: assumed closed (Not functioning or damaged)
◊ Sluice gate: Open — Q_{SG} = 5 m3/s
◊ Overflow over weir crest — Q_{weir} = 21 m3/s
◊ Overflow over main gate Q_{PG} = 4 m3/s
◊ Approx. upstream water level — 201.50 m a.s.l.

The weir crest is of a fix-type at elevation 200.10 m a.s.l. (see numeral 1).

The left side of the weir is connected to a retaining wall (see numeral 2). The main gate opening is 2.70 m wide and 2.40 m high; it is motor-operated. The sluice gate (1.00 m by 1.00 m) is also motor-operated (see numeral 4 and 5).

The submerged sill at el. 198.90 m a.s.l. (see numeral 6) allows sediments to be diverted into the sluice channel (see numeral 7) . The sluice channel has a slope of S = 0.08 which produces a supercritical flow through the sluice channel when the gate is open: it is therefore able to carry a maximum volume of sediments with a minimum of water losses.

A dividing, guide wall (see numeral 8) helps to maintain the flow in the correct direction.

The flood protection wall (see numeral 10) has its top at el. 201.90 m a.s.l. which gives a safety margin of 0.40 m above the maximum possible water level of 201.50 m a.s.l., Which would take care of an emergency such as the impossibility of opening the main gate

The right retaining wall (see numeral 12) directs the flow into the intake. The trash rack of 5.30 m width and 1.25 m height with an inclination of 75°, is fitted with 8 mm x 80 mm steel bars space 25 mm between bars. The rack has to be cleaned manually at regular intervals to avoid clogging with floating material.

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

A remote indicating water level instrument, installed on the right retaining wall(see numeral 16), indicates continuously the water level in front of the intake structure.

Downstream of the intake entrance gate, three rows of stilling racks are provided (see numeral 17) , the function of which is to induce settlement of the coarse granular material carried by the flow into the desilting basin.

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

The desilting basin is designed for a discharge of 3.5 m3/s (see numeral 19). its main characteristics are:

- ◊ L = 32.00 m
- ◊ W = 4.00 m
- ◊ D = 3.50 m
- ◊ V = 0.25 m/s
- ◊ d = 0.25-0.50 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 those given in the example of the water intake for Q = 2.6 m3/s

All operations of the intake can be conducted from a centralized control room (see numeral 20) connected to the intake installation by cables (see numeral 21)

A low tension cable (see numeral 22) provides the power for operation of all instruments and the gates. The gates can, however be opened by hand during emergencies.

A small hand operated water dotation gate, of 0.30 m by 0.40 m, (see numeral 23) takes care of other requirements such as water supply for irrigation and drinking purposes.

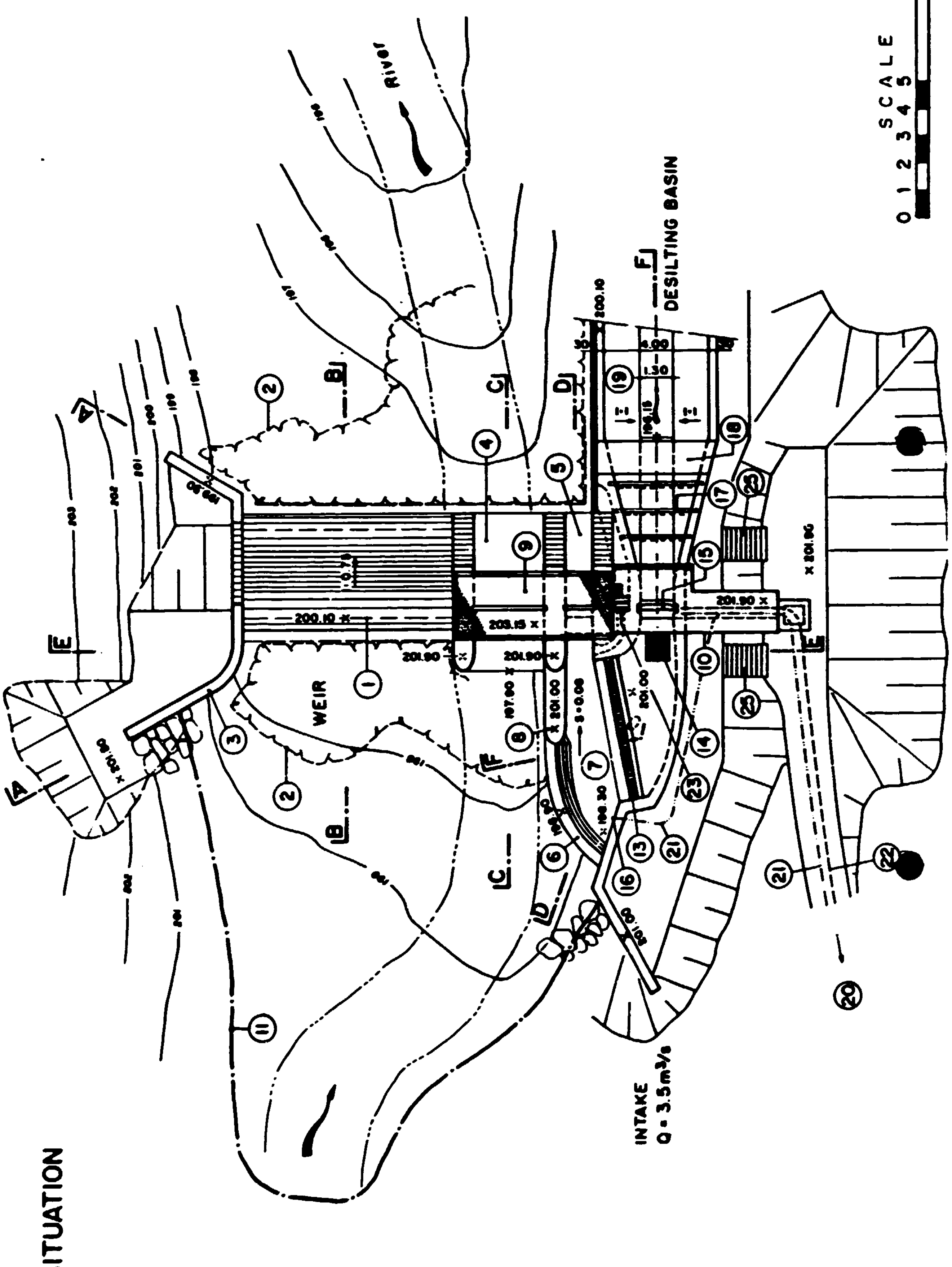
EXAMPLE OF A CONVENTIONAL WATER INTAKE WITH Q = 3.5 m^3/s

LEGEND

1. WEIR CREST El. 200.10 m a.s.l.
2. LIMITS OF THE ROCK SURFACE
3. LEFT SIDE RETAINING WALL
4. MAIN GATE OPENING 2.70 m x 2.40 m (MOTOR OPERATED)
5. SLUICE GATE 1.00 m x 1.00 m (MOTOR OPERATED)
6. SUBMERGED SILL El. 198.90 m a.s.l.
7. SLUICE CHANNEL S = 0.08
8. GUIDING WALL El. 201.00 m a.s.l.
9. GATES OPERATING PLATFORM El. 201.90 m a.s.l.
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 AERATION PIT 0.70 m x 0.70 m
15. ENTRANCE GATE 1.00 m x 1.00 m (WITH MOTOR-CONTROL), ALSO USED AS FLOOD CONTROL.
16. WATER LEVEL
17. STILLING RACKS
18. FOOTBRIDGE.

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
    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. OPERATING BUILDING
21. REMOTE CONTROL CABLE
22. LOW TENSION POWER CABLE
23. DOTATION GATE 0.30 m x 0.40 m WITH MANUAL OPERATION
24. SEAL MADE WITH CLAY
25. STAIR

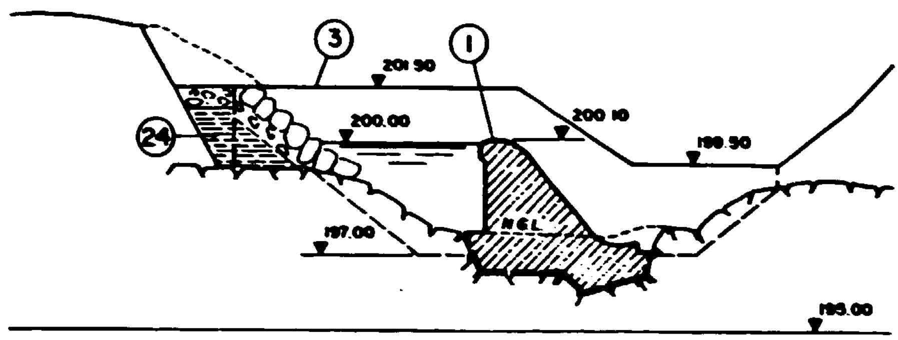
N.G.L.= NATURAL GROUND LINE
R. B. = RIVER BED



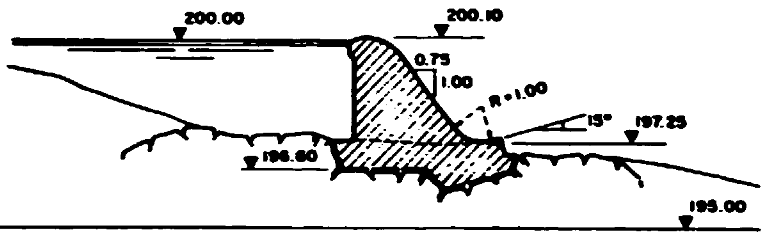
SITUATION

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION			EXAMPLE 3
Engineers + Consultants ZURICH - SWITZERLAND	DRA: DESIGN: APP:	BIEL BU KRU	DRAWING Nr. 40093 - 3 - 1 DECEMBER 1988

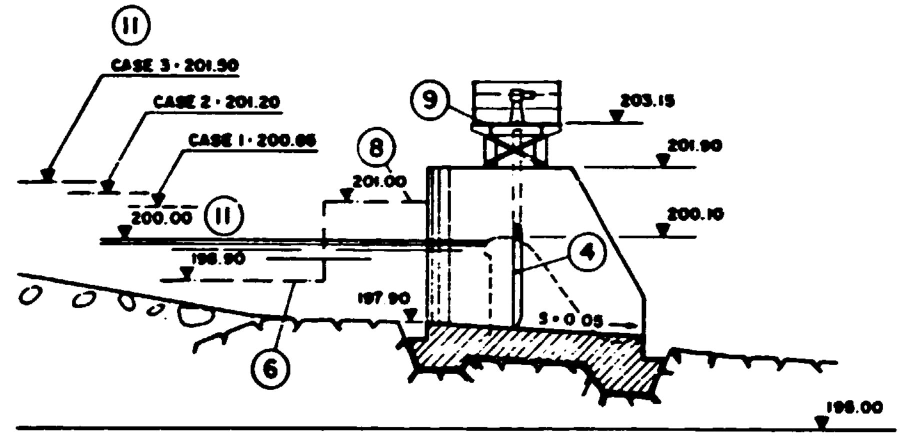
SECTION A - A



SECTION B - B



SECTION C - C



SECTION D - D

SCALE
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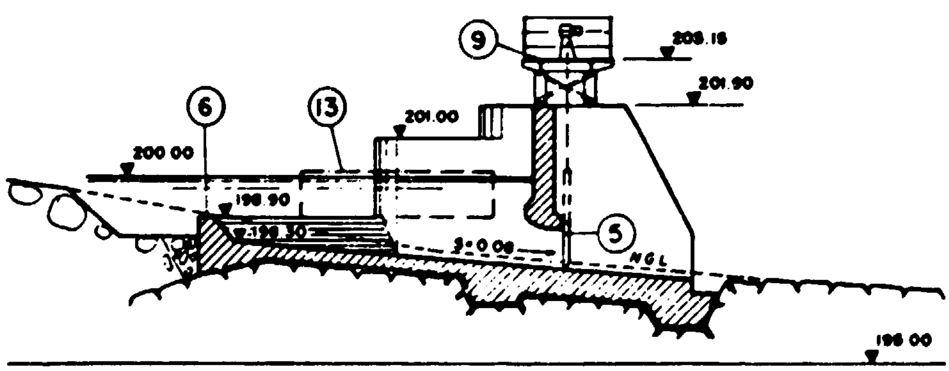
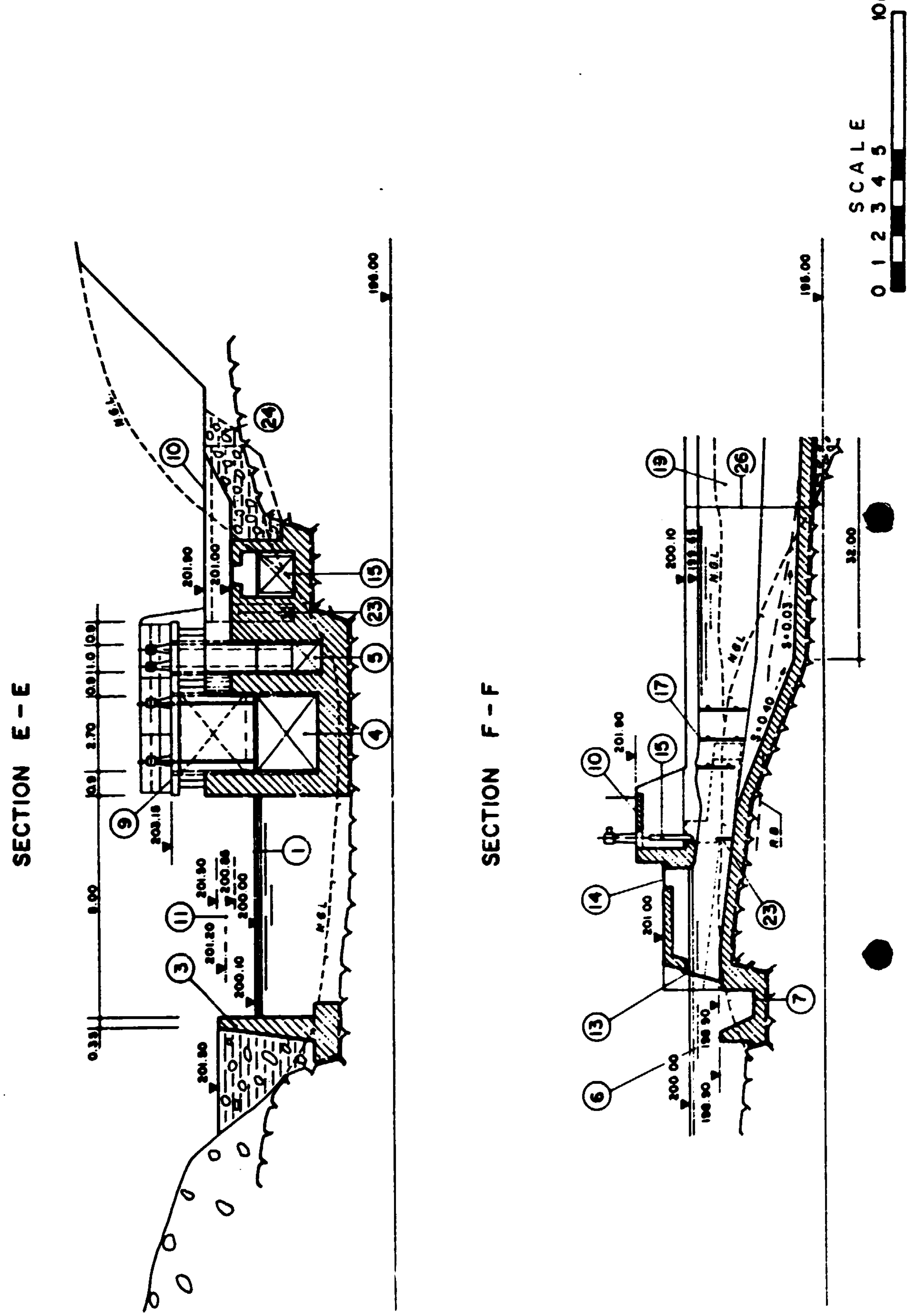


UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION			EXAMPLE 3
Engineers + Consultants ZURICH - SWITZERLAND	DRA: DESIGN: APP:	SRG BU KRU	DRAWING Nr. 40093 - 3 - 2 DECEMBER 1988



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION			EXAMPLE 3
Engineers + Consultants ZURICH - SWITZERLAND	DRA: DESIGN: APP:	BIEL BU KRU	DRAWING Nr. 40093 - 3 - 3 DECEMBER 1988

Engineers + Consultants

EXAMPLE 4

DROP INTAKE

$Q = 1\ m^3/s$

EXAMPLE OF A DROP INTAKE FOR Q = 1 m3/s

An example of a Drop Intake for a design discharge of 1 m3/s is presented in Drawing Nr.40093-4-1. This example has been worked out taking into particular consideration minimum maintenance requirements.

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

As small catchments in mountain areas can produce much higher specific discharges during floods than medium or large catchment areas, 1 m3/s Q_{design} drop intake has been dimensioned, for a 1000-year flood of 12 m3/s as an accepted average for similar projects.

In a mountainous area, with floods of 12 m3/s, a high concentration of sediments is to be expected. Those sediments will have to be separated from the water.

For discharge floods of 4 to 6 m3/s, a volume of 1 to 5 m3 of sediments per day can be assumed, Such values are in accordance with experience gained for other similar projects. This large volume of sediments will normally require at least one flushing a week, and one additional flushing will be necessary immediately after every flood of more than 4 m3/s.

The inclination of the rack is 35°, which gives better separation of clear water from the sediments with diameters greater than 20 mm for discharges less than 3.5 m3/s. For higher discharges, alternative design consideration have to be made.

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

The intake has a small forebay excavated in the rock, to give more regular and turbulence-free approach flow conditions.

On the right, two openings of 0.80 m 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; it is also provided with stop logs slots, thus allowing closure of the intake entrance with two elements each of 0.40 m.

This above intake requires weekly control and when necessary clearing stones and leaves from the rack.

During the 1000-year flood the water will overflow the crest of the weir with a depth of water of 0.70 to 0.80 m.

If there is any indication from the morphological features of the catchment, that flash floods might occur, it is recommended to place the intake on the side of the river, and to foresee an upstream protection wall against the impact of the flood; a wall of gabions may fulfil this purpose.

The desilting plant will be emptied by manual operation about once a week and after every large flood.

After the drop pit, below the rack, the water passes through the entrance and a narrow section (see Drawing Nr. 40093-4-1 numeral 9), limiting any excess flow during floods. The excess flow will be diverted through the lateral spillway constructed for this purpose (see numeral 18). A flood protection wall (see numeral 19) is located at the canal-inlet.

DROP INTAKE AND DESILTING DESIGNED FOR Q = 1m³/s

LEGEND

1. FOREBAY
2. INTAKE SILL ELEVATION 1800.00 m a.s.l., W = 1.80 m
3. TEMPORARY DIVERSION, OPENING SILL ELEV. 1799.25 m a.s.l., W = 0.80 m.
4. STOP LOGS SLOTS
5. STOP LOG PLACING DEVICE
6. TRASH RACK S = 0.70 m, BARS Ø = 24 mm, SPACE 25 mm
7. ENTRANCE CANAL
8. ACCESS AND AERATION PIT
9. NARROWED SECTION 0.65m / 0.65m
10. STILLING RACKS
11. DESILTING BASIN, L = 22 m, W = 2.20 m, Q = 1 m^3/s, d = 0.30 - 2.00 mm, v = 0.25 m/s, W_o = 0.035 m/s
12. HANDRAIL
13. FLUSHING CHANNEL
14. FLUSHING GATE (MANUAL) 0.80m / 0.80 m
15. END SPILLWAY
16. MASONRY
17. FOOT BRIDGE
18. FLOOD SPILLWAY
19. FOOT BRIDGE AND FLOOD CONTROL
20. ADDUCTION CANAL Q = 1 m^3/s, S = 0.0012
21. STOP LOGS
22. AERATION

Engineers + Consultants

EXAMPLE 5

FOREBAY

Q = 1 m3/s

EXAMPLE OF A FOREBAY FOR Q = 1 m^3/s

For the assumed example with a design discharge of 1 m^3/s, a forebay with a surface area of about 60 m^2, an average depth of 2.00 m with a regulation reserve of 0.50 m has been foreseen, (see Drawings No. 40093-5-1 and 5-2).

The minimum depth of 2.00 m has been calculated using Gordon's equation , which gives the minimum depth needed for avoiding vortices with the consequent danger of air entrainment

The equation of Gordon is:

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

where:

d = Depth of water (m)
v = Flow velocity in penstock (m/s)
D = Diameter of Penstock (m)

For Q = 1 m^3/s, a velocity of approximately 3.50 m/s and a minimum penstock diameter of 0.60 m (which is the minimum that allows internal inspection) the minimum depth necessary becomes 1.9 m or in round figures 2.00 m .

The minimum forebay area of 60 m2 is in accordance with experience gained with other similar projects (see Table No. 9-1).

With the above dimensions the minimum volume in the forebay becomes 120 to 150 m3, with a regulation reserve of 0.50 m which is a safe value based on extensive experience obtained elsewhere.

The inlet to the penstock is fitted with a trash rack , which for a discharge of 1 m3/s requires a minimum gross area of 1.70 m2, to which corresponds to a gross velocity of 0.59 m/s.

The trash rack is made of rectangular steel bars.80 mm x 8 mm, with separations between the bars of 25 mm, giving a net area of approximately 1.30 m^2. (As a rule of thumb, the net area should be about 75% of the gross area for this type of small intake), The net water velocity is approx. 0.78 m/s.

The lateral emergency spillway has a regulation reserve of 0.50 m and a minimum additional freeboard of 0.25 m. In an extreme situation, overflow will occur at a level of 0.75 m above that of the emergency spillway.

In the case of overflow, and assuming that at the same time the turbine is out of service (1 m3/s), the canal will continue to supply water (another 1 m3/s). With a safety reserve of 0.5 x Q (or 0.50 m3/s), the total overflow is 2.5 m3/s

Assuming a depth of water over the spillway of 0.50 m , and knowing that:

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

or

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

a spillway length of 5.00 m is obtained, with a factor of safety of 1.25

The spillway coefficients calculated using the data given in Table 6.1, with a depth of water of 0.50.m

For the calculation of the minimum water depth in the forebay Gordon's formula can again be used i.e.:

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

this time for a minimum design discharge of o.50 m^3/s, and:

D = 0.60 m (diameter of penstock)
F = 0.283 m2 (area of penstock section)
V = 1.77 m/s (velocity of water in penstock)

A value of d_{min} of 1.00 m is thus obtained

For the narrow valley for which for this example applies, it is not possible to provide flow regulation for more than about half-an-hour upstream of the penstock, because of its natural configuration.

Where a flater areas would be available it would be possible to provide a reserve volume in the forebay for 1 - 1.5 hours of peak generation.

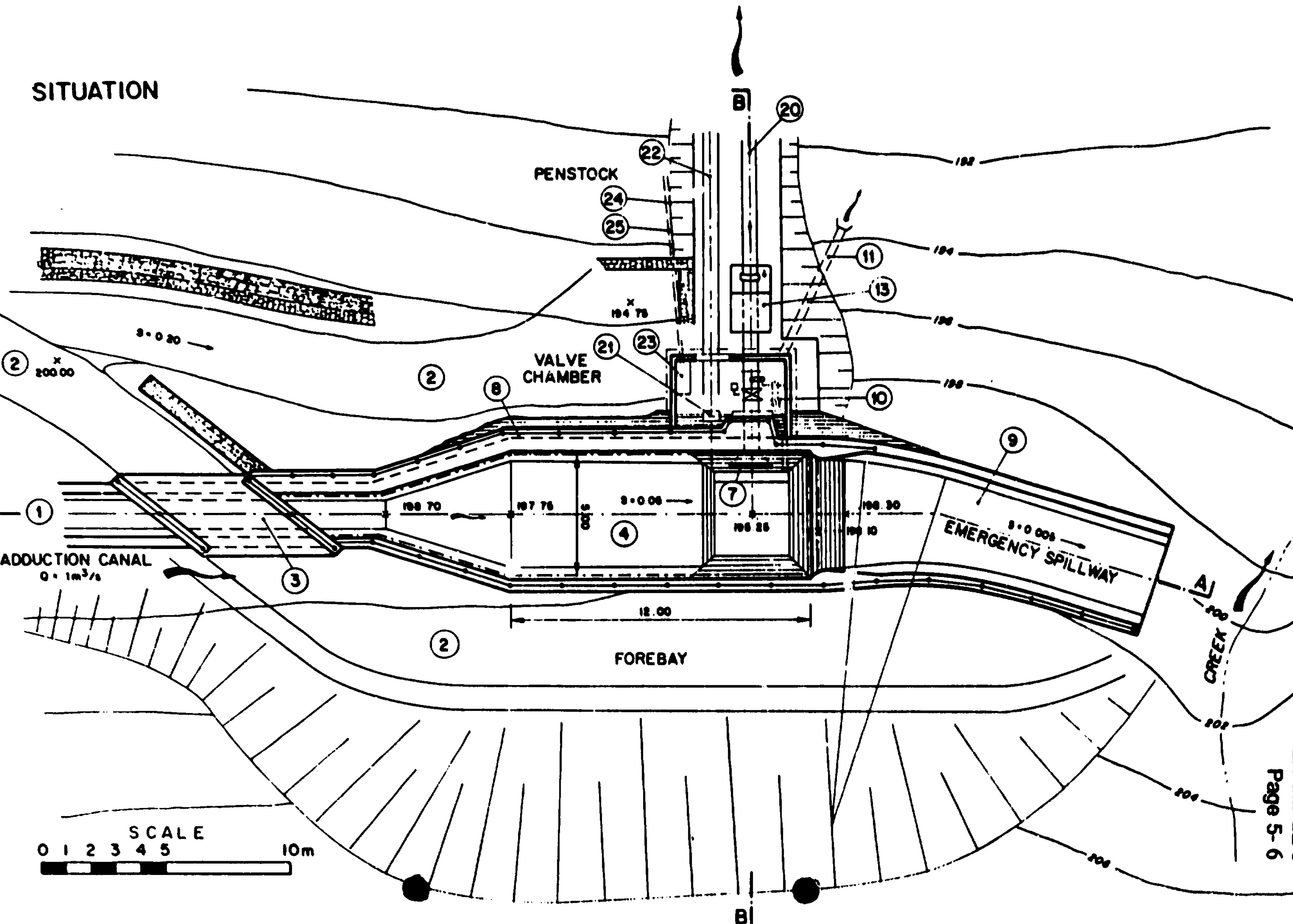
In some isolated plants, energy is supply only during certain hours, thus allowing the production of energy to be increased during such periods. Enough pondage volume should however be available in the forebay or in another type of reservoir.

FOREBAY FOR Q = 1 m3/s

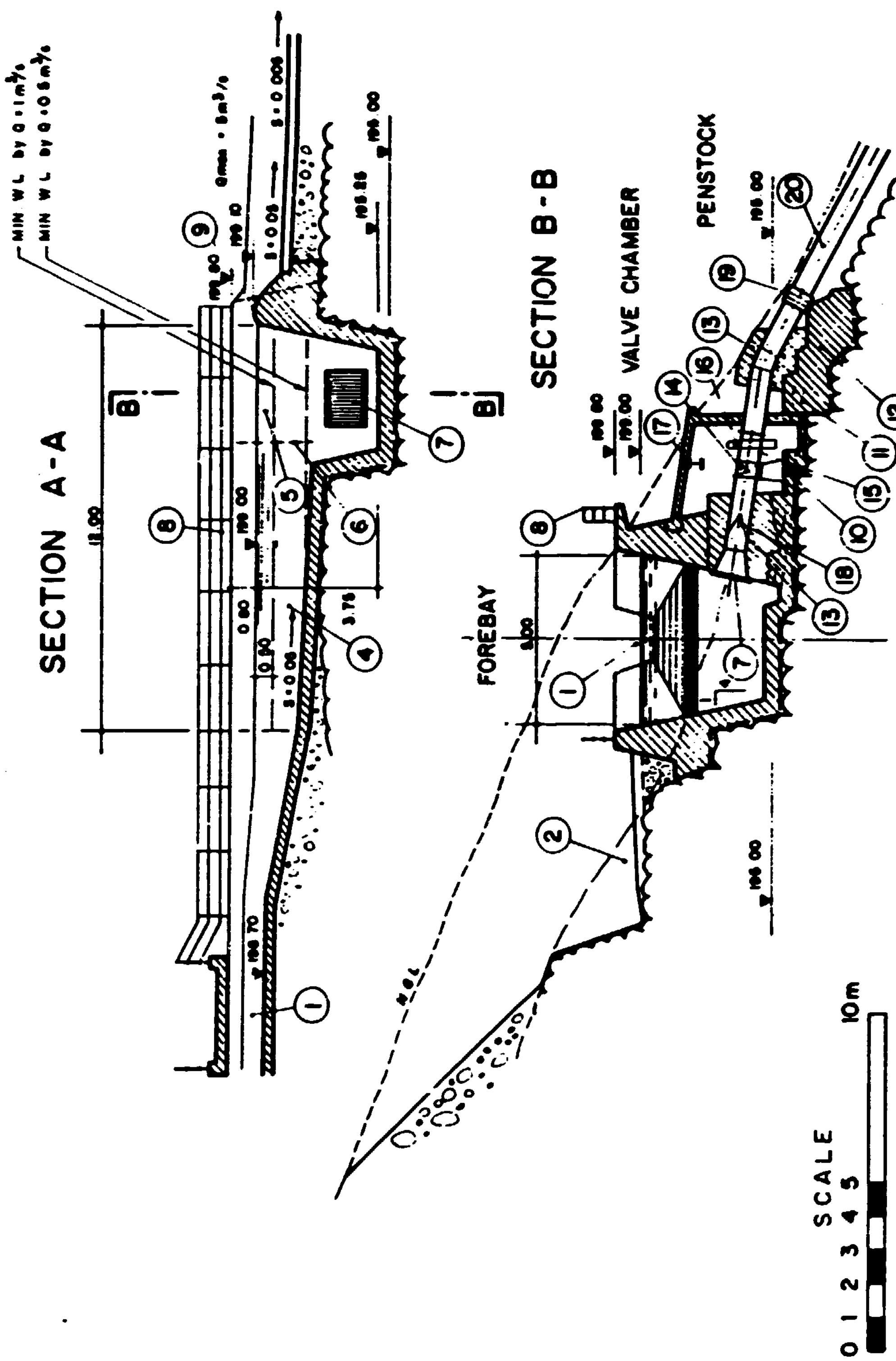
LEGEND

1. ADDUCTION CANAL Q = 1 m3/s
2. ACCESS ROAD
3. BRIDGE
4. FOREBAY
5. REGULATION VOLUME, SURFACE ELEV. 199.00 m a.s.l. 5 m. x 12 m
6. WATER LEVEL CONTROL
7. INTAKE TRASH RACK 1.70 m x 1.00 m
8. FOOT BRIDGE WITH HANDRAIL
9. EMERGENCY SPILLWAY
10. SLUICE VALVE D = 200 mm
11. EVACUATION PIPE D = 400 mm
12. ANCHOR BLOCK FOUNDATION
13. CONCRETE WITHOUT REINFORCEMENT
14. AUTOMATIC BUTTERFLY VALVE D = 500 mm
15. REMOVABLE PIPE AND FLANGE
16. AIR RELIEF VALVE
17. RAIL FOR ERECTION-WINCH
18. STEEL TRANSITION PIECE (OR SLEEVE)
19. EXPANSION JOINT
20. STEEL PIPE D= 200 mm
21. WINCH
22. TRACK (FUNICULAR)
23. ELECTRICAL AND REMOTE CONTROL PANEL
24. ELECTRIC CABLE
25. CABLE FOR REMOTE CONTROL

SITUATION



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 5
Engineers + Consultants ZURICH - SWITZERLAND	DRA: ZU DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 5 - 1 DECEMBER 1988



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION			EXAMPLE 5
Engineers + Consultants ZURICH - SWITZERLAND	DRA:	SRG	DRAWING Nr. 40093 - 5 - 2
	DESIGN:	BU	DECEMBER 1988
	APP:	KRU	

EXAMPLE 6

FOREBAY

$Q = 2.6\ m^3/s$

EXAMPLE OF A FOREBAY FOR Q = 2.60 m3/s

An example of a forebay for a design discharge of 2.60 m3/s is presented in Drawing No. 40093-6-1, with the following characteristics:

Intake type with slide gate of max. 950 mm x 950 mm, with electrically operated, with water-filled gate shaft and side emergency overflow. The gate shuts down automatically for excess velocity in the penstock; the gate can also be closed from the powerhouse. This solution provides an easy erection as well as maintenance and repair.

Dimensioning of the main parts has been made in a similar way as for the forebay for 1 m3/s presented in the previous example 5.

The adduction canal (see numeral 1) has been calculated for a minimum discharge of 2.60 m3/s entering the forebay at elevation 198.15 m a.s.l. (see numeral 3), The water surface is 5.80 m wide and 18.00 m long, the bay having a reserve margin for water oscillations of 0.75 m.

The intake trash rack is 1.80 m high by 2.40 m long, with its centre 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 diameter (see numeral 8) has been foreseen for manual operation. This valve allows periodical cleaning of the forebay.

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

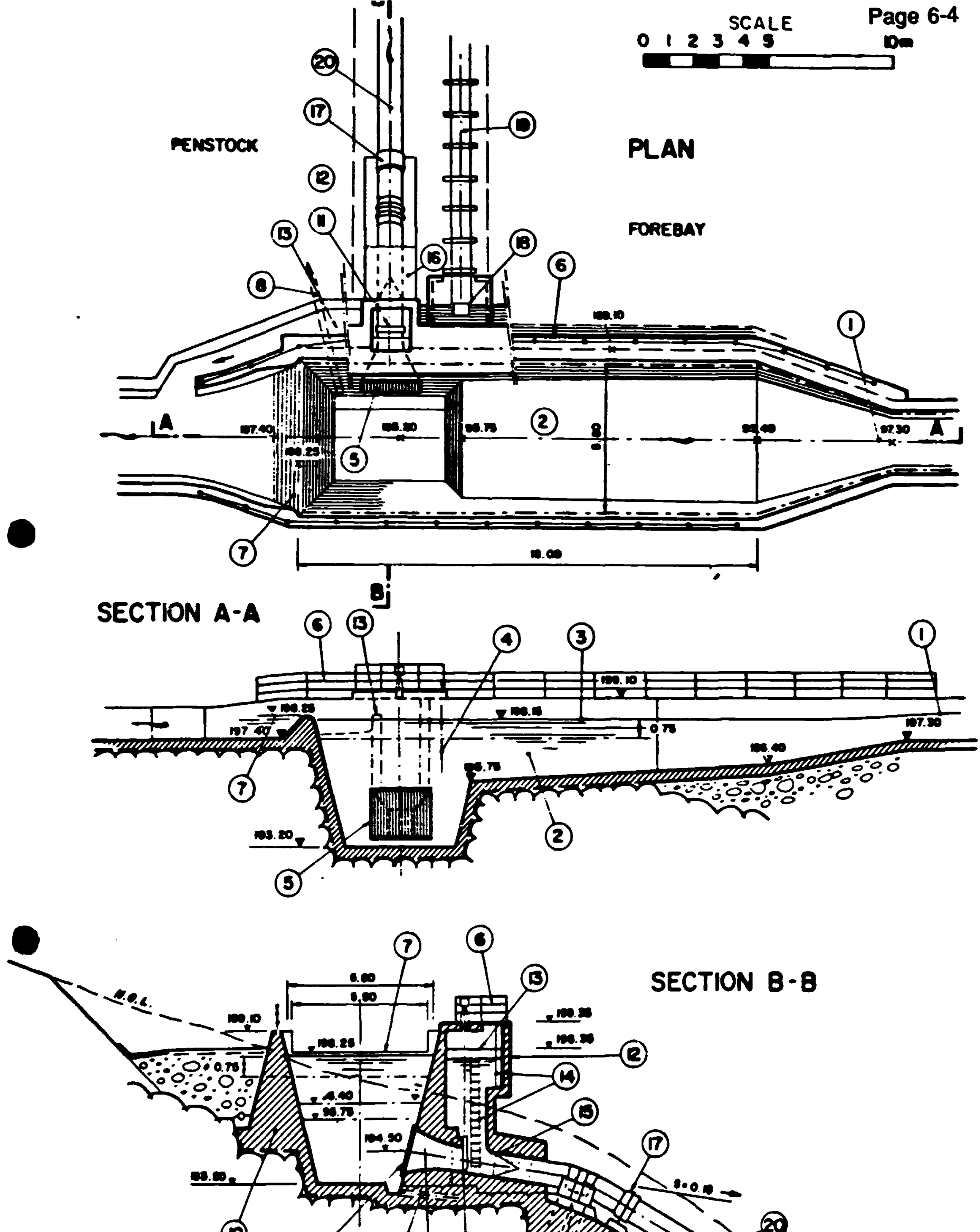
The 950/950 mm head slide gate (see numeral 11) is reached from the trash rack through the intake mouth. It will be electrically operated from the platform at elevation 199.35 m a.s.l.

The gate shaft (see numeral 12) serves for aeration of the penstock in case of turbine shut-down; water can pass over the emergency overflow (see numeral 13) . The penstock is designed with a diameter of 950 mm (see numeral 20).

FOREBAY FOR Q = 2.6 m^3/s

LEGEND

1. ADDUCTION CANAL Q = 2.60 m^3/s
2. FOREBAY
3. FOREBAY OPERATING VOLUME WITH WATER LEVEL AT ELEV. 198.15 m a.s.l., 5.80 m WIDTH AND 18.00 m LENGTH, WITH 0.75 m FOR POSSIBLE WATER OSCILLATIONS.
4. WATER LEVEL OPERATING GEAR
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. STEEL CONDUIT INLET
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
Engineers + Consultants ZURICH - SWITZERLAND	DRA: DESIGN: APP:	BIEL BU KRU	DRAWING Nr. 40093 - 6 - 1 DECEMBER 1988

Engineers + Consultants

EXAMPLE 7

GENERAL LAYOUT OF

HYDROELECTRIC PLANT AND PENSTOCK

$Q = 1\ m^3/s$

GENERAL LAYOUT OF A HYDROELECTRC PLANT WITH A PENSTOCK FOR $Q = 1\ m^3/s$

The general layout with a longitudinal section of a hydropower plant is presented in Drawing No. 40093-7-1; it SHOWS a complete installation for a small power plant DESIGNED FOR Q = 1 m3/s.

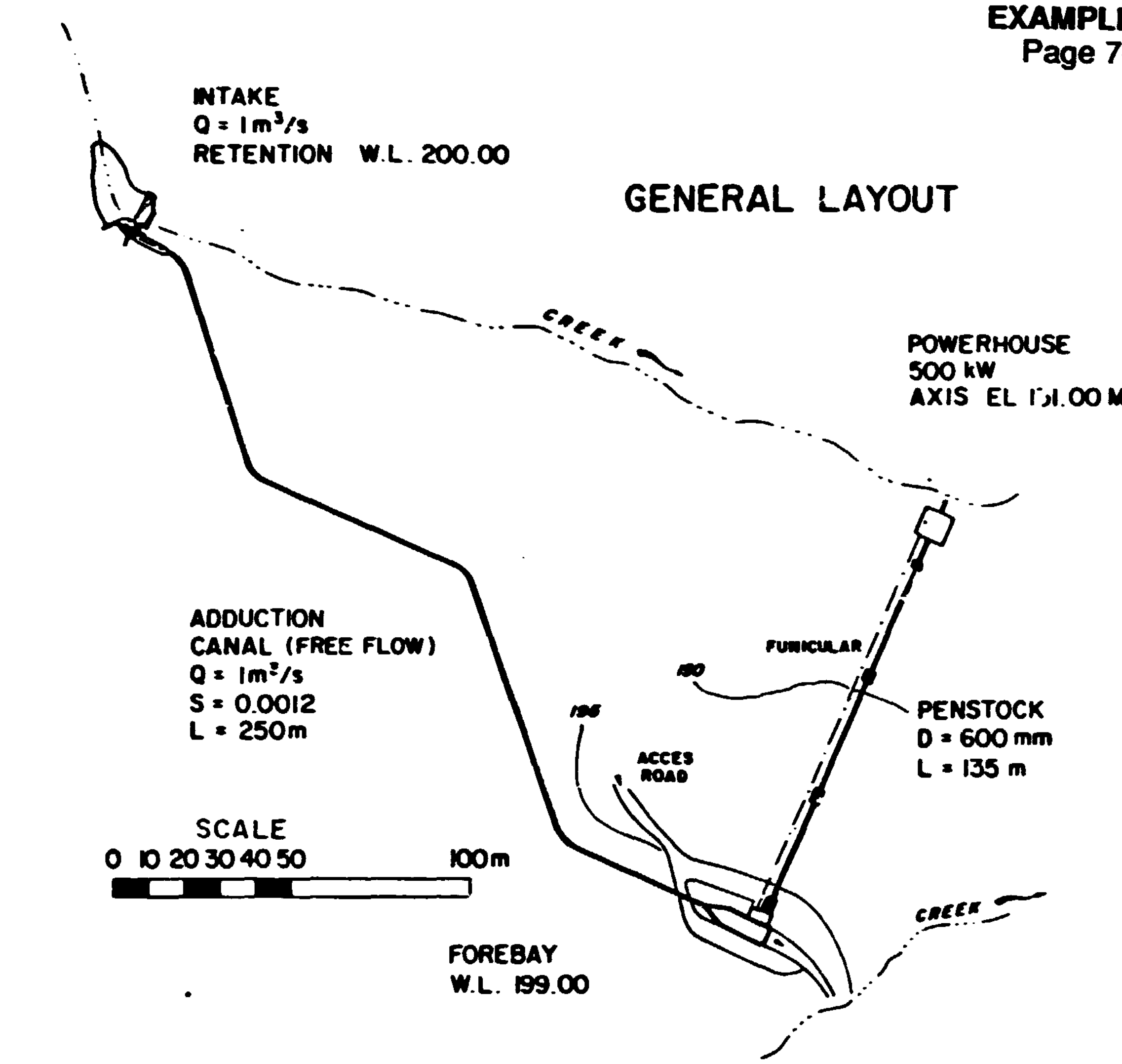
The choice of the intake and powerhouse locations is of prime importance, as the ratio of length to head must be as small as possible.

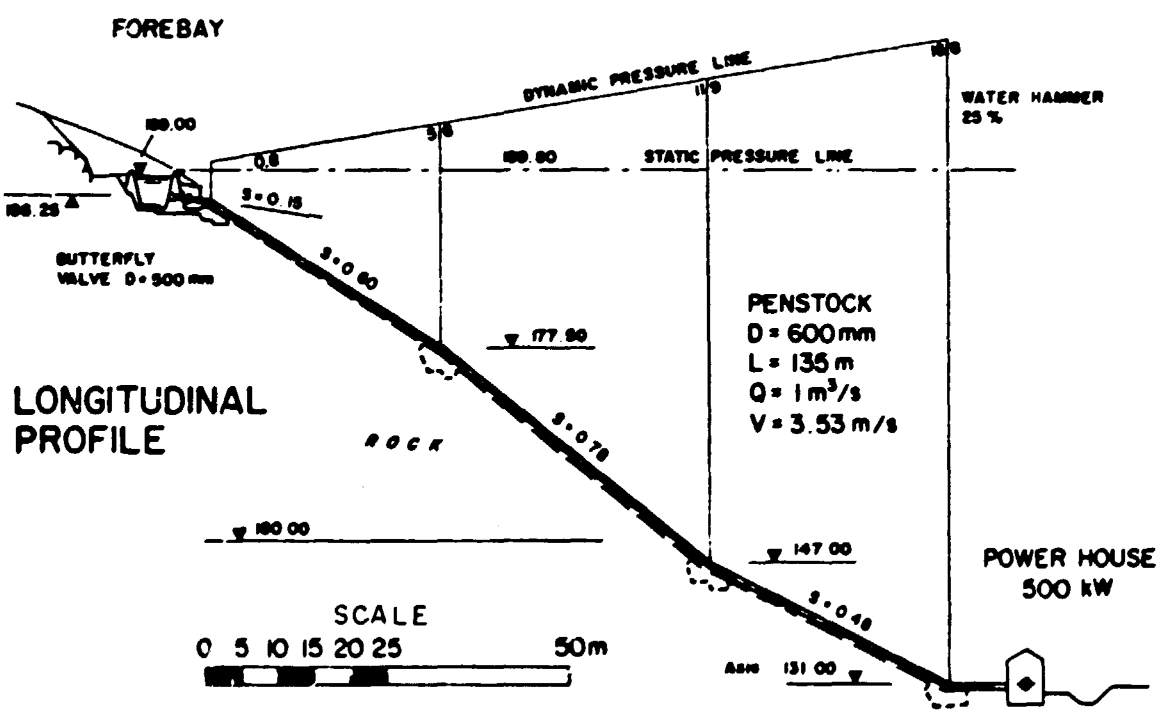
The location of the intake requires particular attention; it will be placed at the highest possible elevation, taking into account the location on a bend of the river, under the best possible geological conditions, and at a narrow cross-section. The adduction canal should be able to follow a topographically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

This general layout assumes the retention and the intake structures to be at elevation 200.00 m a.s.l. The water is then conveyed by a free-flow adduction canal to the forebay at elevation 199.00 m a.s.l. and from there to the powerhouse THROUGH A penstock, 135 m long and 600 mm diameter.

The powerhouse with an installed capacity of 500 kW, is at 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 has to be remembered here that in all cases the dynamic effect should be computed and it must be ascertained whether a surge chamber is required or not) . The velocity flow in the penstock is 3.53 m/s, which can be considered as safe. The penstock between the forebay and the powerhouse should be as short as possible.





UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION			EXAMPLE 7
Engineers + Consultants ZURICH - SWITZERLAND	DRA: DESIGN: APP:	ZU BU KRU	DRAWING Nr. 4C093 - 7- 1 DECEMBER 1988

Engineers + Consultants

EXAMPLE 8

PENSTOCK

$Q = 2.6\ m^3/s$

EXAMPLE OF A PENSTOCK FOR Q = 2.6 m3/s

An example of a penstock for a Q = 2.6 m^3/s is presented in Drawing No. 40093-8-1.

The first part of the penstock is assumed to be on sound rock foundations, the second part on well-consolidated alluvial soils.

A forebay with a butterfly valve of 800 mm diameter controls the entrance to the penstock. It could be replaced the butterfly valve by a slide gate valve .

Two types of anchors blocks are shown. That for the upper part,founded on rock, represents the most favourable condition for a penstock. The second, for the lower part founded on overburden of well consolidated alluvial soil, is only possible if the geological conditions of the natural soil are stable. If it is not possible to find a foundation in rock, and the saddles are to be founded on overburden; a slope of maximum S = 0.44 should not be exceeded.

Where no rock foundation is possible the pipe must be completely embedded in concrete to ensure a low bearing pressure on the overburden. For the total length of the pipe, a slope of not more than S = 0.60 is allowable.

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

EXAMPLE 9

WELDED STEEL BURIED PENSTOCK

Q = 0.90 m^3/s and Q = 2.50 m^3/s

EXAMPLE OF A WELDED STEEL BURIED PENSTOCK WITH Q = 0.90 m3/s and Q = 2.50 m3/s

The example shown on Drawing 40093-9-1 is for a welded steel buried penstock with Q = 0.90 m^3/s built in a favourable longitudinal profile of mostly gravel ground. With exception of a road crossing where concrete embedding was used, the penstock is buried in an excavated and thereafter normally filled trench. However for the lower part below the road crossing, with a 53% slope inclination, the fill had to be supported by small wall structures

A man hole of 600 mm diameter was placed near the road providing an adequate access for the inside revision of the lower penstock part.

Although only an inclination angle of 25° (with the horizon) upstream of the power house was apparently required for the penstock; an actual angle of 32.2° was necessary for design and manufacture of the penstock on the work shop, because of three dimensional deviation in plan (see sketch on the following drawing).

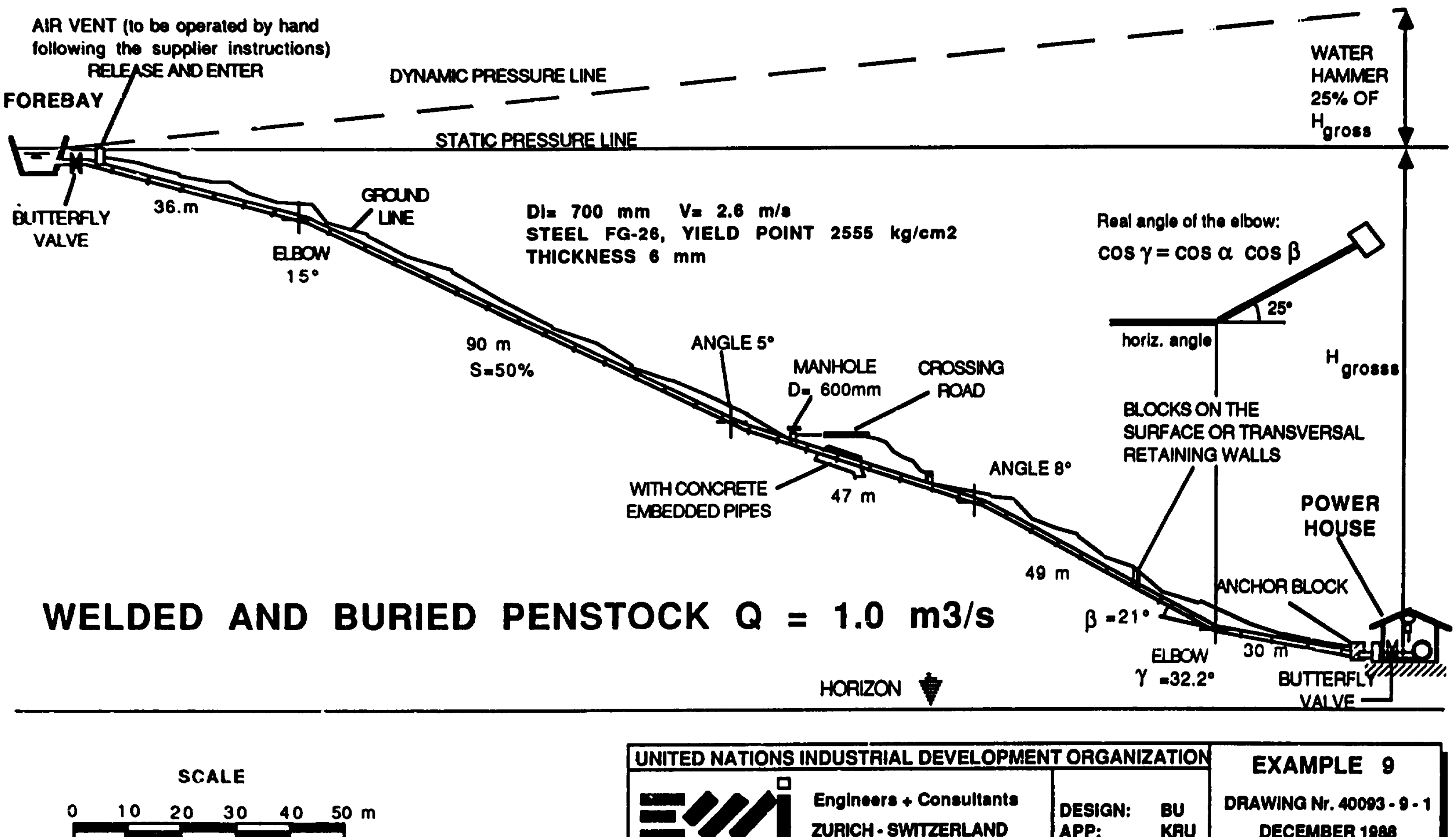
As a second example Drawing 40093-9-2 illustrates a welded and buried penstock for a Q = 2.5 m^3/s built in a straight line, with normal longitudinal slope inclination.

For both cases, civil works were carried out from the upstream to downstream direction but the erection of the steel pipe; was made in the opposite direction, is i.e., starting in the power house and ending in the forebay.

The provision of a temporary funicular was recommended; it was built for carrying out the civil works, and in particular, the erection of the steel pipes.

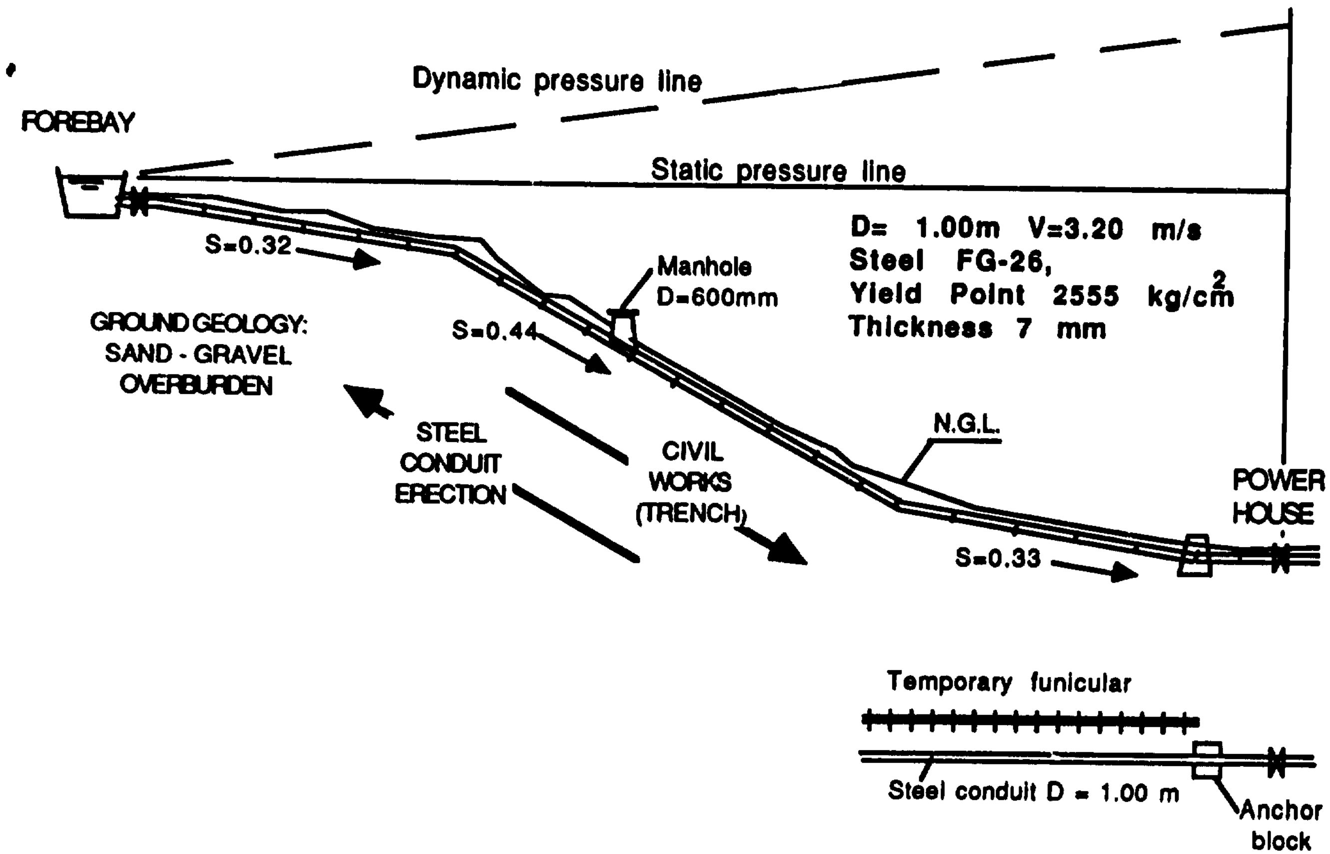
For the second example the ratio of the relation L/H is high which means that the water velocity is within the upper allowable range.

The details of the trench excavation for both cases are shown on Drawing 40093-9-3, those of the manhole on Drawing 40093-9-4 and lastly those relating to the welding of the pipes on Drawing 40093-9-5



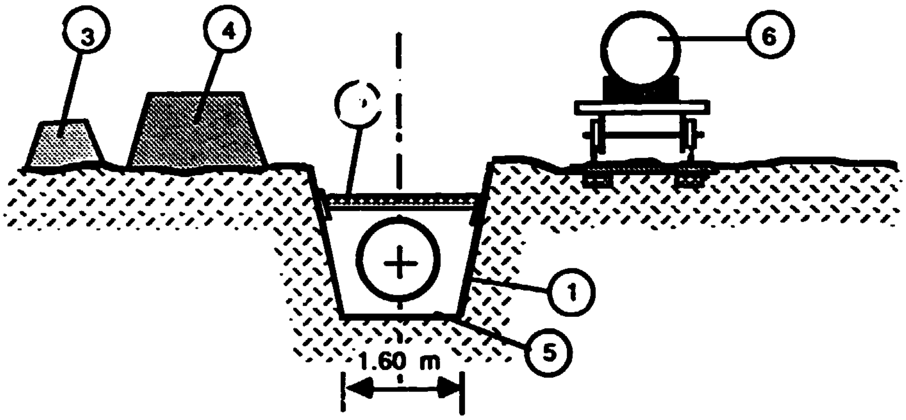
WELDED AND BURIED PENSTOCK Q = 1.0 m3/s

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 9
Engineers + Consultants ZURICH - SWITZERLAND	DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 9 - 1 DECEMBER 1988



WELDED AND BURIED PENSTOCK Q = 2.5 m^3/s

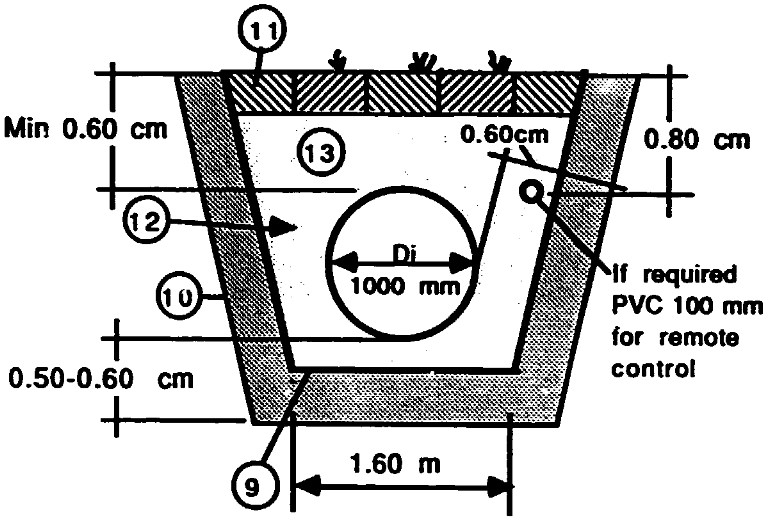
UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 9
Engineers + Consultants ZURICH - SWITZERLAND	DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 9 - 2 DECEMBER 1988



1. Trench excavation
2. Shoring or bearing where required, temporary
3. Humus deposit (temporary)
4. Border (strip) deposit of the excavated material
5. Bottom preparation and drainage of rainwater and infiltration water
6. Track for pipes transport

TRENCH EXCAVATION AND PREPARATION FOR PIPES TRANSPORT

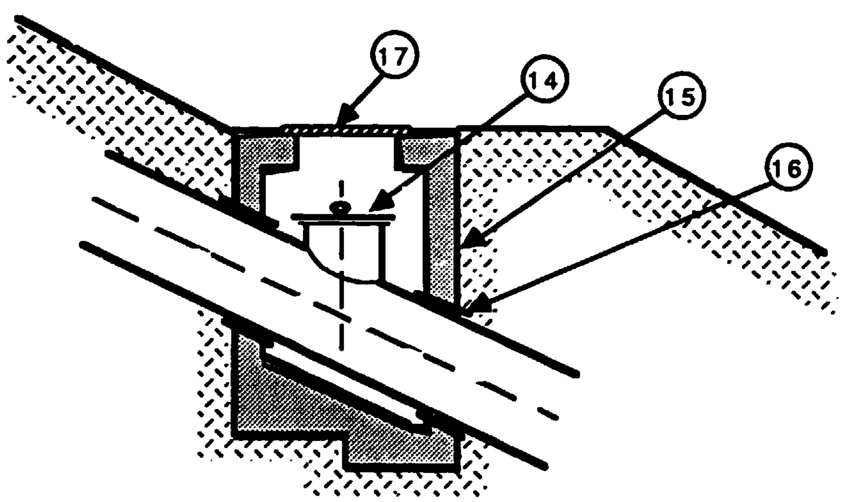
UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 9
Engineers + Consultants ZURICH - SWITZERLAND	DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 9 - 3 DECEMBER 1988



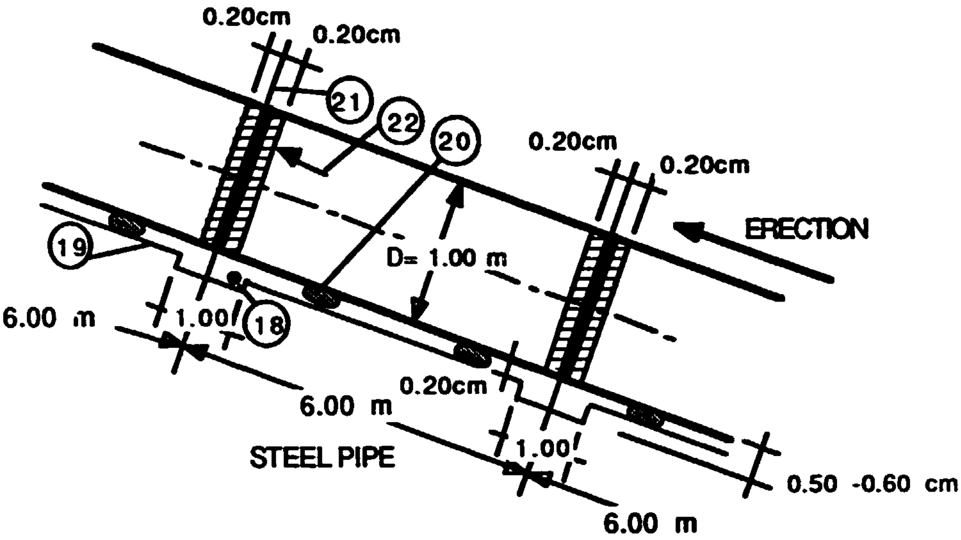
DETAIL OF TRENCH

9. Trench excavation
10. Recess excavation for welding
11. Humus 20-30 cm
12. Fill with earth, sand, gravel d= 0-0.50 mm, compacting around the pipe.
13. Earth fill compacted
14. Screwed manhole D=600 mm
15. Manhole chamber
16. Roofing felt
17. Access cover

DETAIL OF TRENCH



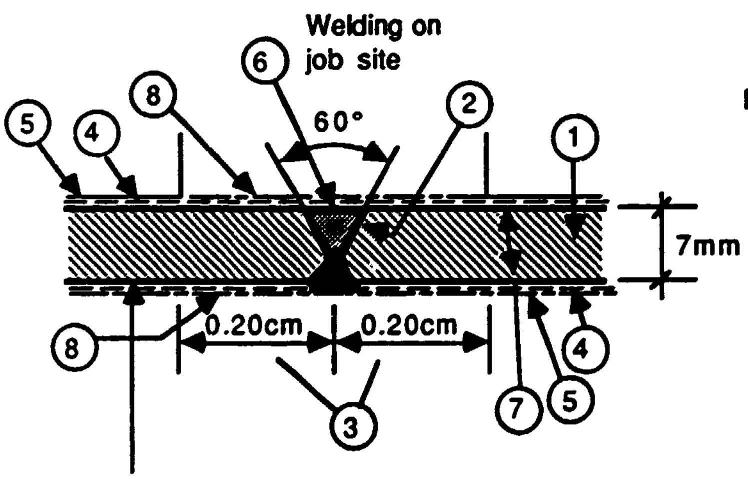
DETAIL OF MANHOLE
FOR INSPECTION
OF THE PENSTOCK



ERECTION

18. Recess for welding
19. Bottom of trench
20. Jute bags with sand
21. Job site Welding
22. Job site Connection Painting on both sides of the joint

ERECTION OF THE PIPE LINE



Di = 600 to 700mm — only outside weld, inside grinding after welding

Di = 1000mm — inside and outside welding. inside griding to the flat surface

PIPE ENDS' WELDING AND PAINTING

1. Pipe wall
2. Chamfer 2 x 30°
3. Paint free sections at both ends
4. Sandblasting SA.3
5. 3 x TAR-EPOXY total 0.5 mm
6. External weld seam to grind after welding, also inside surface
7. To brush, to clean
8. Final paint layer inside & outside 3 x TAR-EPOXY Total 0.5 mm to take particular care of the outside surface

EXAMPLE 10

PRESSURE TEST

EXAMPLE OF A PRESSURE TEST

This example relates to the details and explanations required for the pressure test of the penstock described in the Example 9.

The pressure test is carried out after the welding and all painting of the conduit have been completed. As a rule of thumb a pressure equal to 1.5 times the value of the static pressure is applied. The test pressure should be maintained for a minimum of 24 hours.

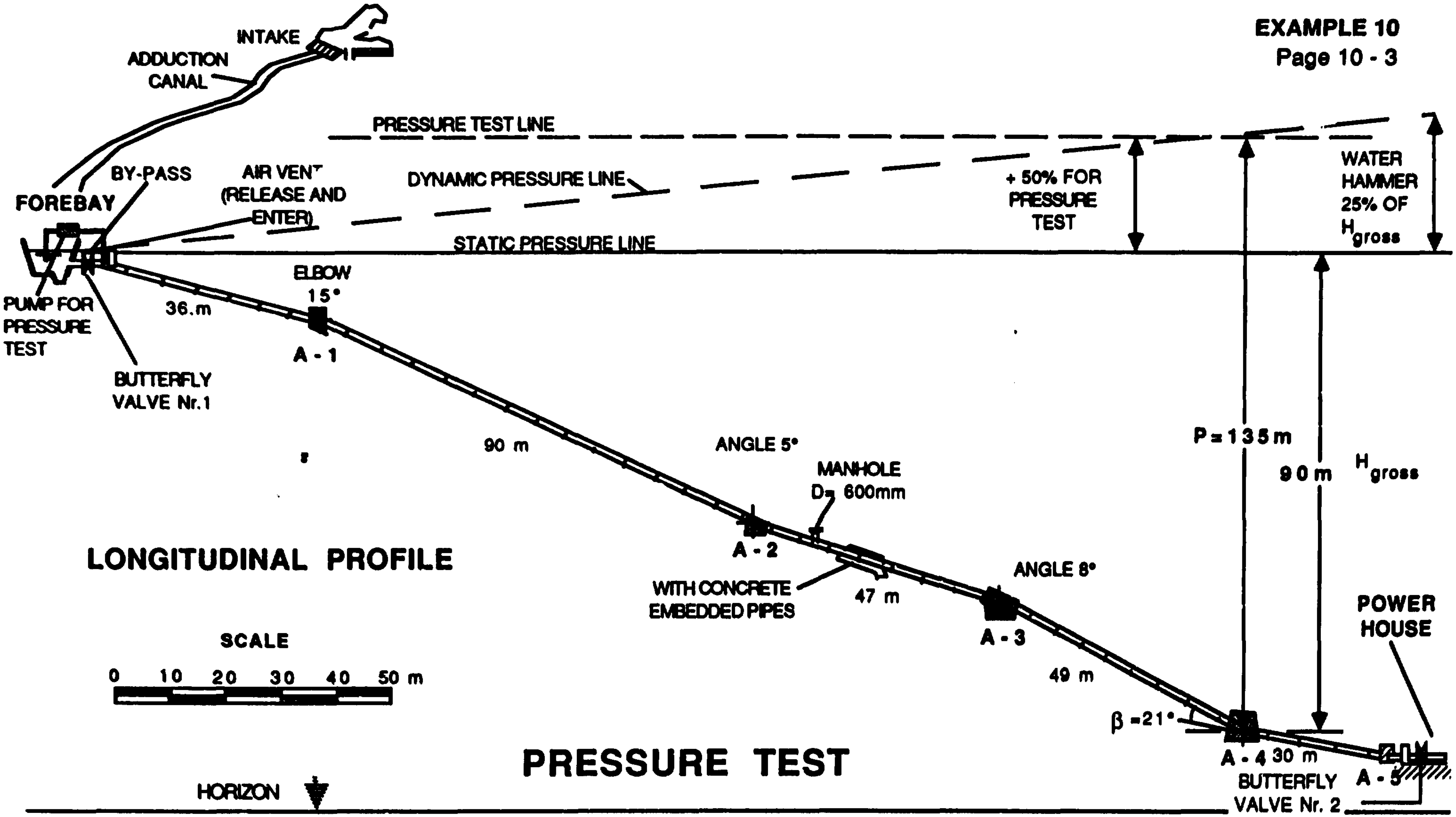
The pressure test is carried out with the penstock completely, welded and painted in the trench, but not yet buried. After all the valves and man holes are checked to be closed, the penstock is filled with water through the by-pass on the upper part of the penstock with the help of a hand pump. Once the pipe is filled with water all welded parts are carefully checked, reporting any anomalies, leaks, etc., observed.

If no leakage is observed during a period of not less than 24 hours, the air vent can be opened allowing air entrance and removal of the water in the penstock. Should however, any leakage be observed, the defective spot has to be immediately repaired and thereafter the penstock pressure test repeated.

A final painting check for anticorrosive protection should be made: repairs of any damaged areas of painting should be carried out before backfilling the steel pipe into the excavated trench.

After backfilling the trench, the soil above the pipe should be protected in an adequate way with vegetation, on both sides of the alignment of the pipeline.

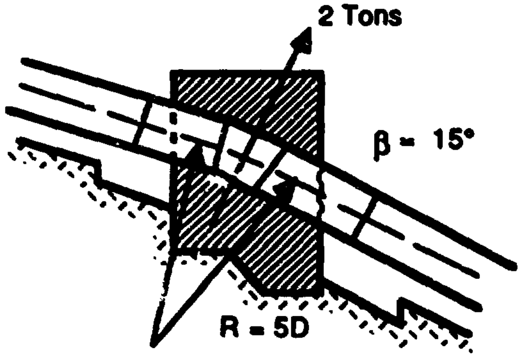
Examples of various types of anchor blocks are given as a reference.on drawings 40093-10-3 and 4



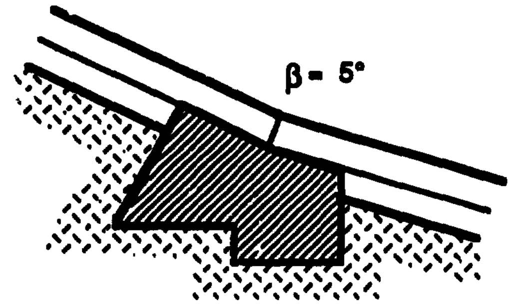
LONGITUDINAL PROFILE

PRESSURE TEST

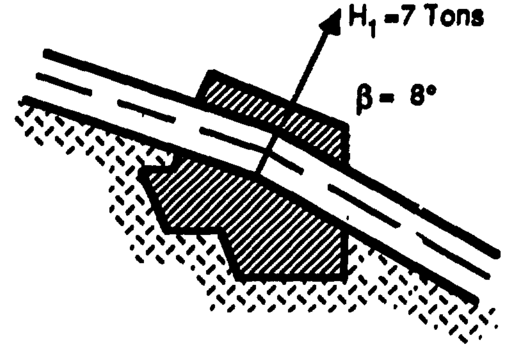
UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 10
Engineers + Consultants ZURICH - SWITZERLAND	DESIGN: BU APP: KRU	DRAWING Nr. 40093 -10 - 1 DECEMBER 1988



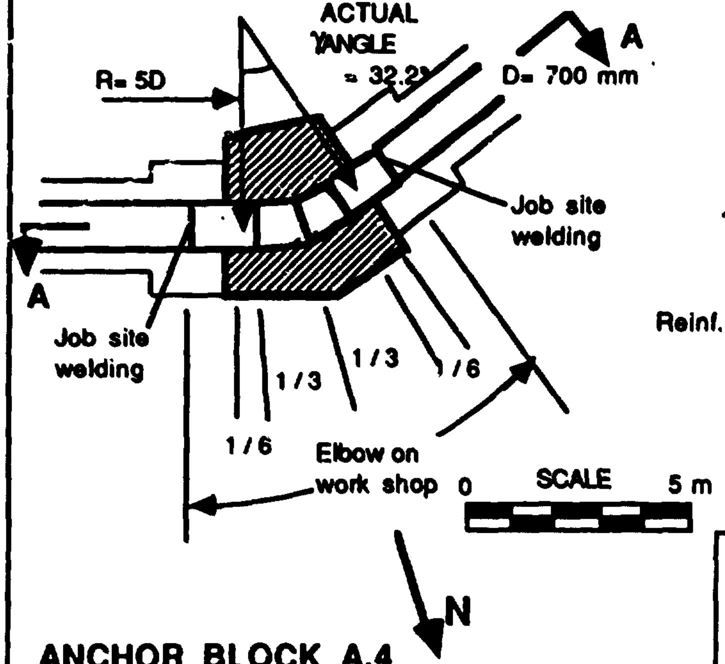
ANCHOR BLOCK A.1



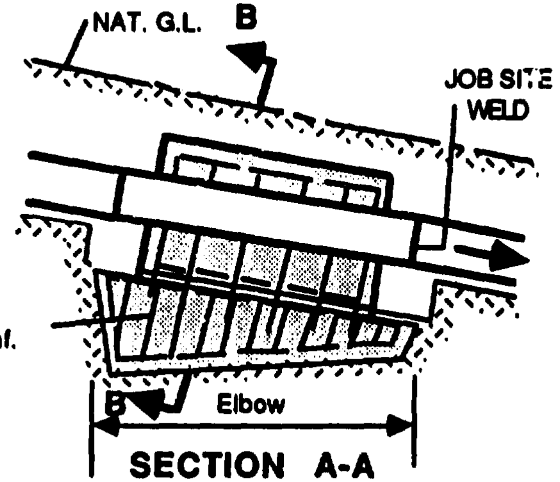
ANCHOR BLOCK A.2



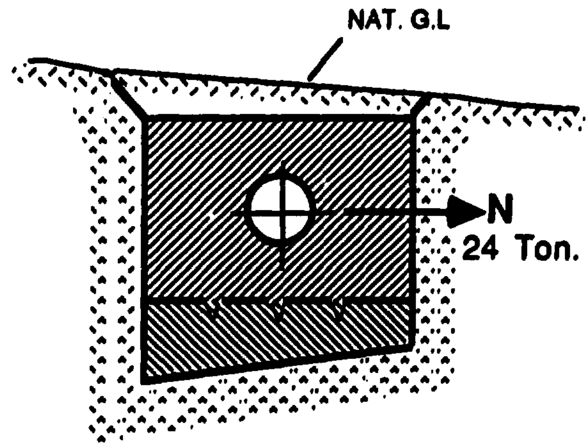
ANCHOR BLOCK A.3



ANCHOR BLOCK A.4

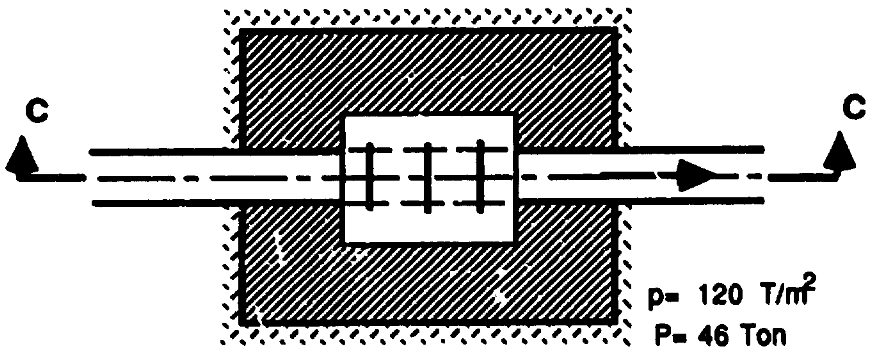


SECTION A-A
(Longitudinal Section)

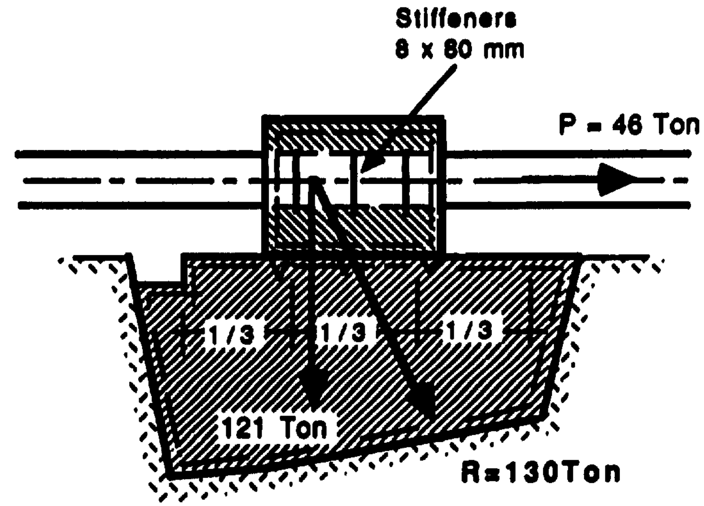


SECTION B-B
(Cross Section)

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 10
Engineers + Consultants ZURICH - SWITZERLAND	DESIGN: BU APP: KRU	DRAWING Nr. 40093 - 10- 2 DECEMBER 1988



ANCHOR A.5
(PLAN)



SECTION C-C
(LONGITUDINAL SECTION)

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 10
Engineers + Consultants ZURICH - SWITZERLAND	DESIGN: BU APP: KRU	DRAWING Nr. 40093 -10 - 3 DECEMBER 1988

PRESSURE TEST

1. THE WHOLE CONDUIT IS ERECTED, BUT NOT YET BURIED,ALL ANGLES OF THE PENSTOCK ARE EMBEDDED IN CONCRETE, THE ANCHOR A.5 (COMPUTED FOR DYNAMIC PRESSURE) IS ERECTED.

2. THE BUTTERFLY VALVES No.1 AND No.2 ARE CLOSED

3. FILLING OF THE ADDUCTION SYSTEM UPTO THE FOREBAY

4. MANHOLES TO BE CLOSED

5. SLOWLY FILLING OF THE PENSTOCK THROUGH THE BY-PASS NEAR THE VALVE No.1 WITH 50 l/s AND WITH THE HELP OF ONE HAND PUMP UNTIL THE 135 M PRESSURE IS REACHED

6. CHECKING OF ALL JOB SITE WELDINGS STARTING FROM THE POWERHOUSE

7. MAINTAIN THE PENSTOCK FILLED DURING A 24 HOUR PERIOD

8. IF NO LEAKAGE IS OBSERVED, CLOSE THE VALVE No.1 AND, OPEN THE AIR VENT TO ALLOW FOR AIR ENTRANCE AND THEREAFTER EMPTYING THE PENSTOCK.

9. CAREFUL BACKFILLING OF THE WHOLE TRENCH AROUND THE STEEL PIPE.

Engineers + Consultants

EXAMPLE 11

PENSTOCK WITH SURGE TANK

$Q = 0.90\ m^3/s$

EXAMPLE OF A PENSTOCK WITH Q = 0.90 m3/s

1. Generalities

This is an example of a power plant located in the vicinity of a high snow-bound mountainous country. The first part of the conduit crosses mostly pasture-ground. The last third, however passes through broken country with a steep cross-section. In steep places it can be exposed to snow slips and avalanches, and therefore, an open conduit trench would fill rapidly with snow and ice. A buried conduit was therefore chosen.

For the implementation of the plant, following steps were undertaken:

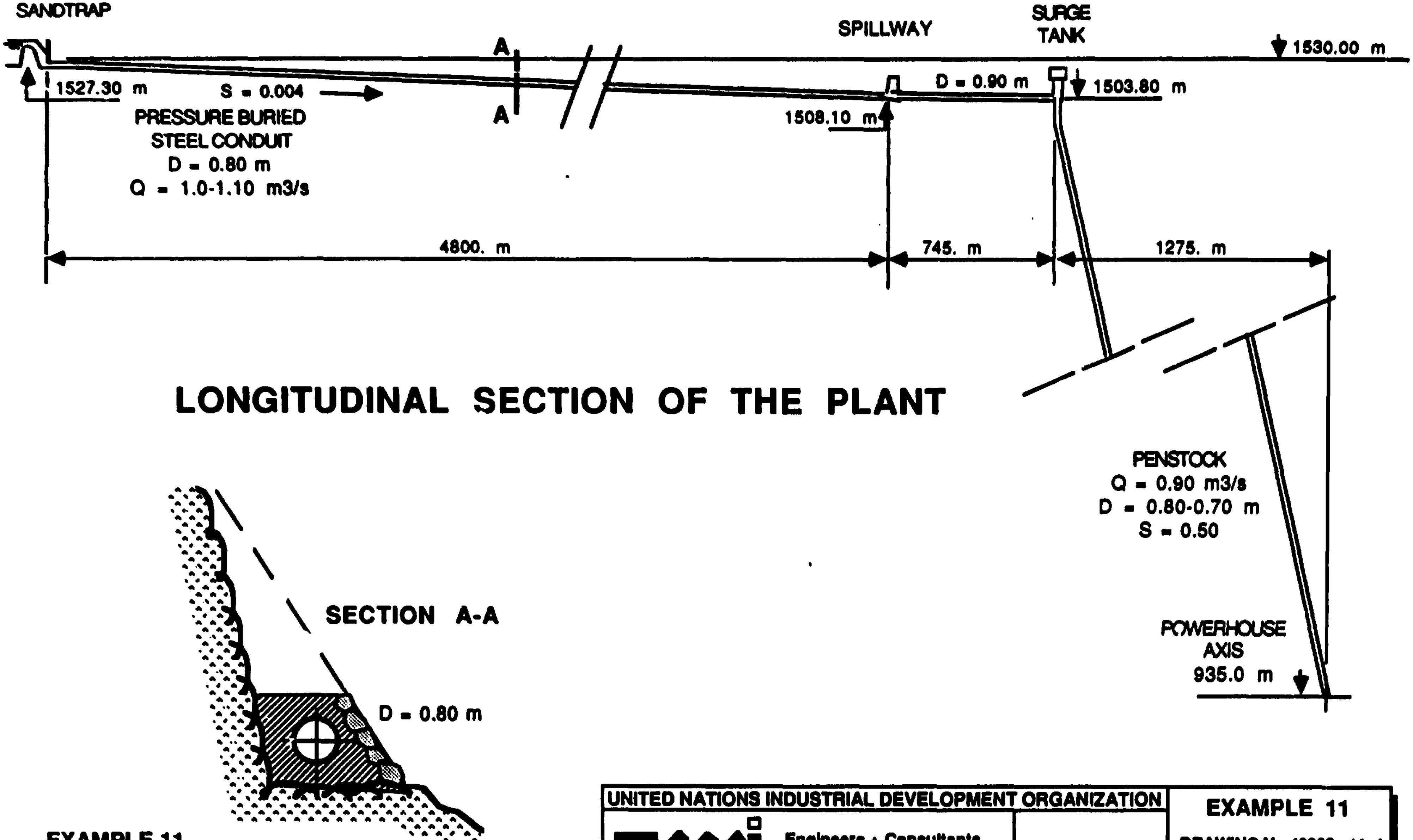
a) Choice of and decision on the ideal tracing of the penstock
b) Choice of the type and location of the surge tank
c) Choice of the intake site, taking into consideration a minimum slope of 0.004 from the surge tank and a favourable location on the river site.

2. Surge tank characteristics

The spillway was placed 745 m upstream of the surge tank precisely where the overflow water can be evacuated in a creek (whereby, the necessary protection measures for the downstream people must be taken into consideration).

The overflow crest represents the static level, at which, in case of a shut-down of the plant, oscillating water from the surge tank, added to the water flow Q from the intake, will overflow over the spillway into the creek. After a certain number of oscillations, the static level will return to a normal condition.

An automatic electrical water level control device is installed in the surge tank, transmitting the recorded values to the power house control board.



LONGITUDINAL SECTION OF THE PLANT

SECTION A-A

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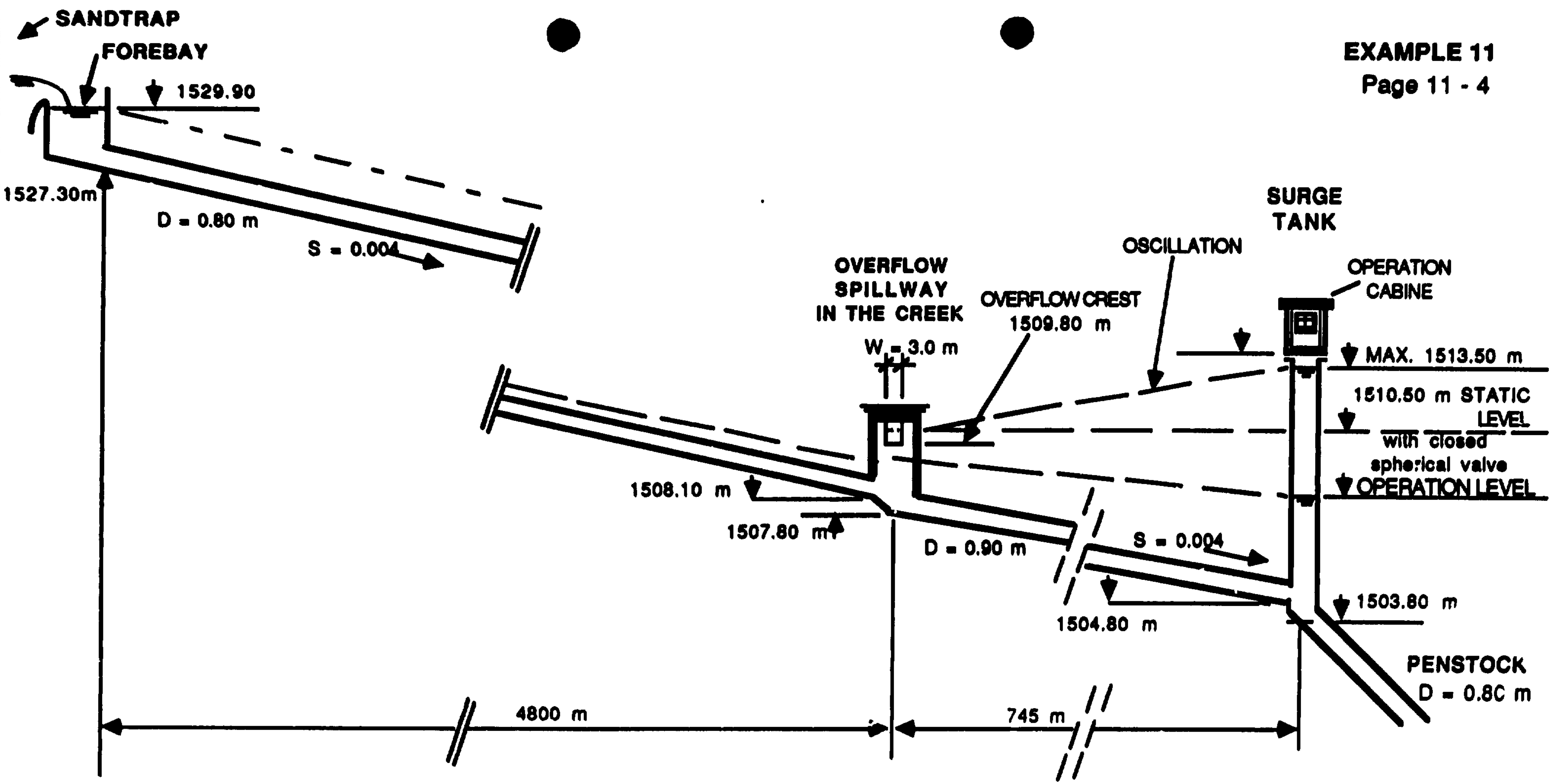

SCHEME OF THE SURGE SYSTEM

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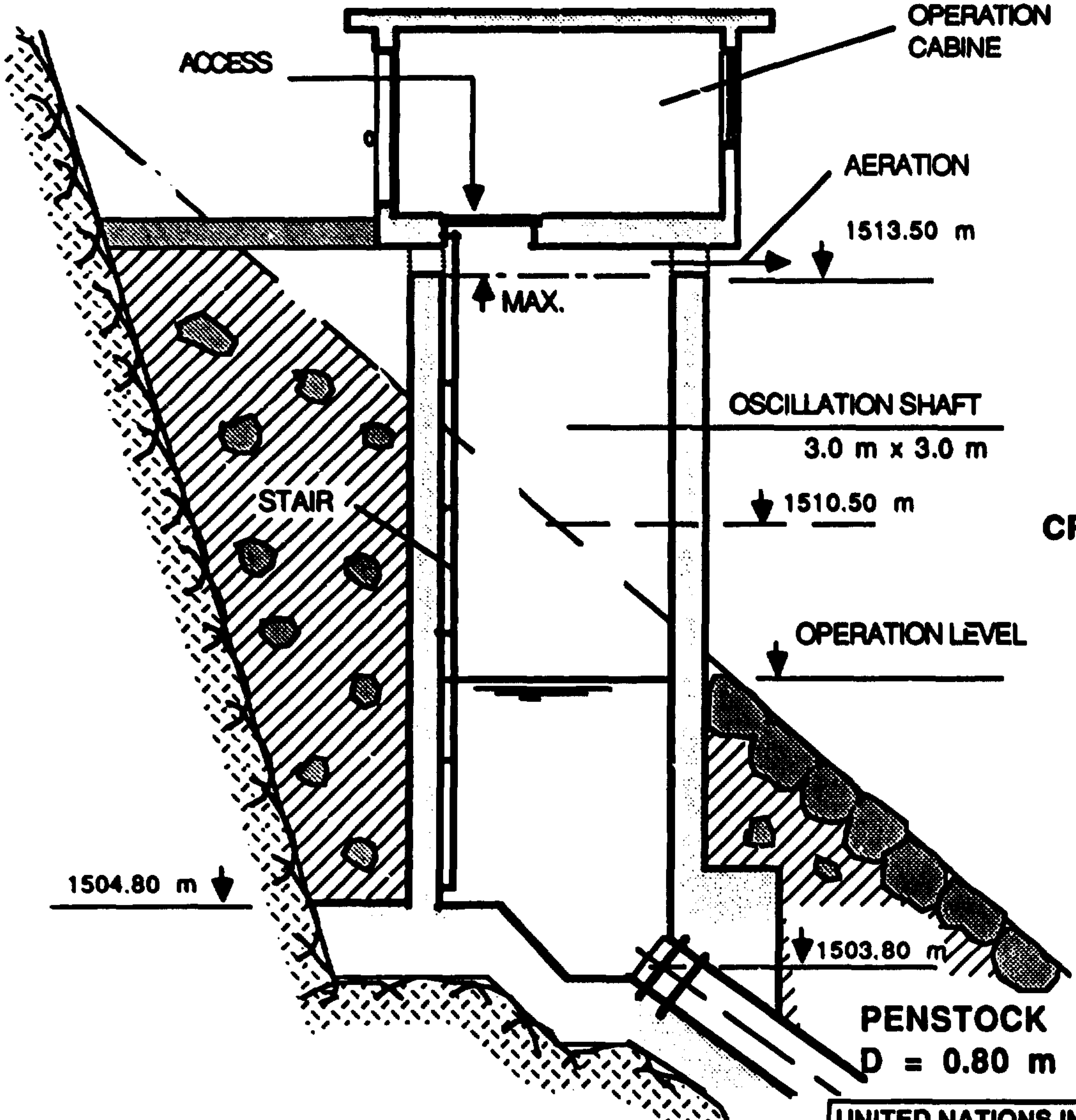
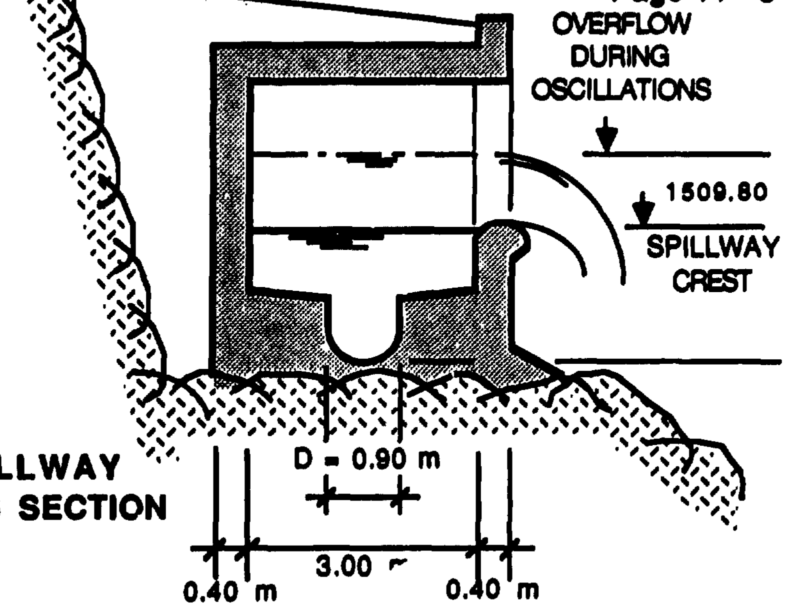
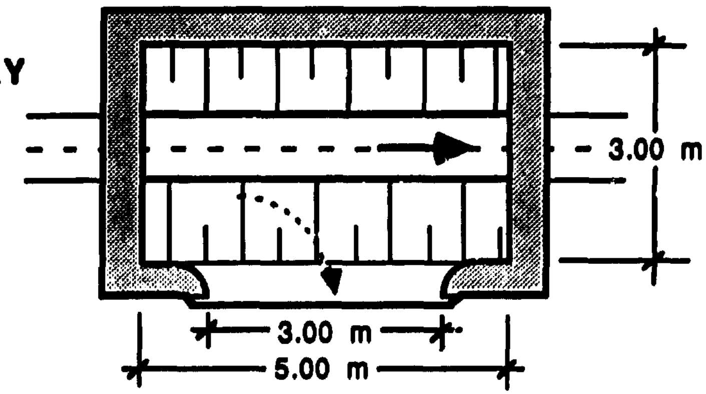


SURGE TANK



SPILLWAY CROSS SECTION

SPILLWAY PLAN



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION		EXAMPLE 11
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EXAMPLE 12

PENSTOCK WITH SURGE CHAMBER

$Q = 3.00\ m^3/s$

EXAMPLE OF A PENSTOCK WITH Q = 3.00 m3/s

This example relates to an intake connected to a retention with 4.0 m range for water level fluctuations.

The pressure conduit passes mainly through a forest with several bends, but with a regular longitudinal slope of 0.015, well adapted to the terrain. The pressure penstock itself has a relative small slope of s = 0.173. The surge tank system is located in the prolongation of the penstock at the trifurcation 4 shown on Drawing 40093-12-1. The complete system is buried. The lower part of the tank with a D = 3.00 m is placed in the alignment of the penstock, its upper end is below the minimum operation water level (piezometric). From there a conduit D = 0.90 m connects the lower with the upper part with a D = 3.00 m ; the bottom of the surge tank is located 4 m higher than the maximum retention level at the intake.

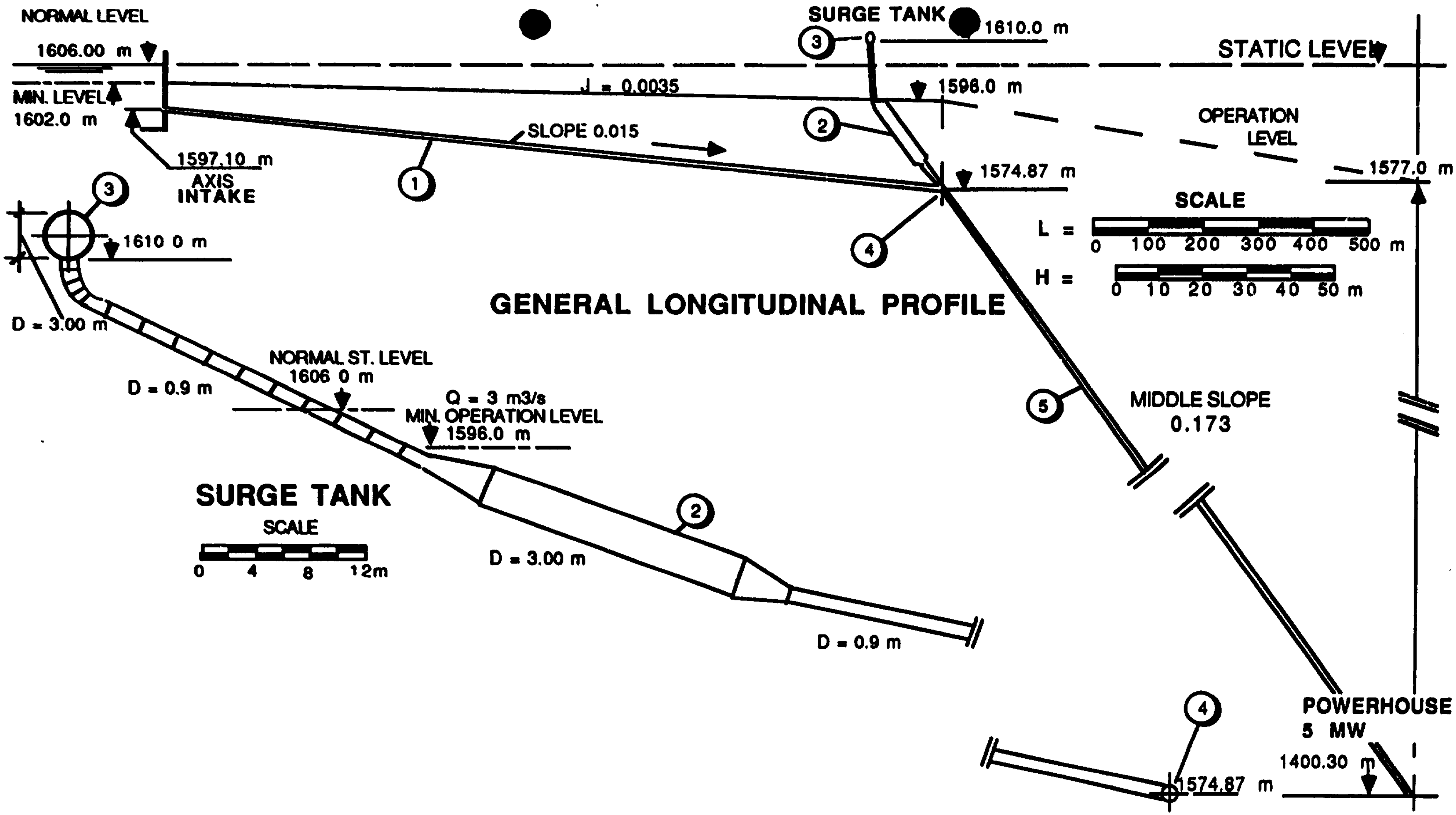
The entire conduit is coated inside and outside with an anticorrosive protection. The various pipe sections are large enough for allowing regular inspections; for that purpose, screwed manholes are provided at regular intervals along the conduit.

The upper chamber of the surge tank, made of steel is protected by a wall of reinforced concrete allowing, a ready check of its outside anticorrosive protection.

During periods of maximum oscillation of the water level, the highest level will reach a high of 40 cm below the top of the tank. One aeration pipe for inlet and outlet air is placed on the top of the tank.

As an alternative more usual solution, a circular or rectangular basin built in reinforced concrete could be considered instead of a surge tank. The basin should be fitted with a high wire mesh fence. Where trees are located in the vicinity of the tank, a roof consisting of a light steel supporting structure covered with wire mesh should be foreseen as protection.

Owing to severe winter conditions, the entire installation of penstock and surge tank are earth covered..



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EXAMPLE 12
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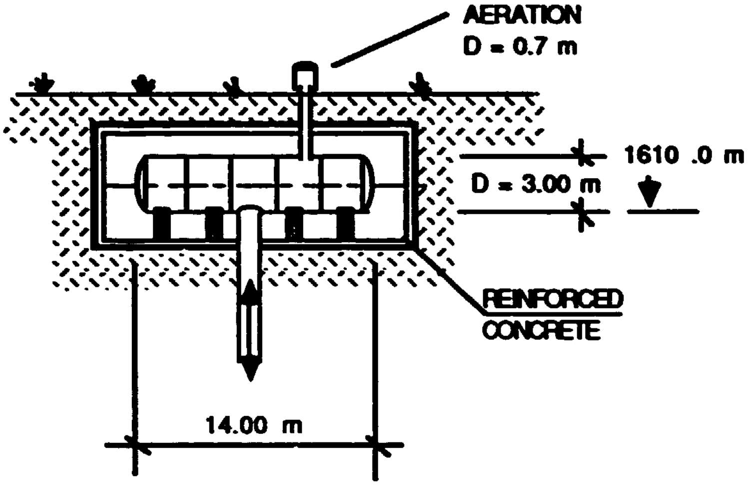
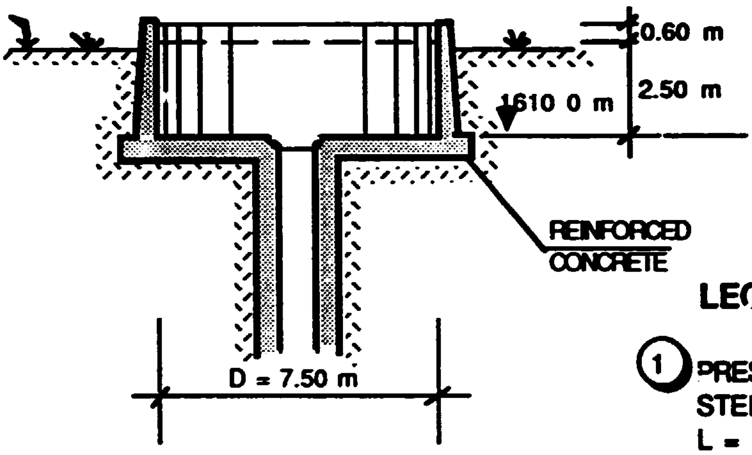


STEEL TANK



REINFORCED CONCRETE
OPEN CHAMBER

GENERAL INDICATIONS:

CATCHMENT AREA IN THE ALPS: 51.4 km2

PRODUCTION 20 000 kWh

POWERHOUSE:
TURBINE FRANCIS Q = 3 m3/s
GENERATOR: 5 MW (cos ϕ = 0.8)
6000 kVA
FREQUENCY 50 Hz
SPEED 750 r.p.m.

REMARK:

THE LOCATION AND TYPE OF THE SURGE TANK DEPENDS ON THE CONFIGURATION OF THE TERRAIN

LEGEND:

1 PRESSURE STEEL CONDUIT
STEEL ALTO 37, THICKNESS 6mm
L = 1460 m, D = 1.20 m, Q = 3.0 m3/s

2 LOWER CHAMBER
D = 3.0 m

3 UPPER CHAMBER:
ALTERNATIVE 1: STEEL TANK D = 3.0 m
THICKNESS 8 mm
PLACED IN REINF. CONC.
CHAMBER AS PROTECTION

ALTERNATIVE 2: REINFORCED CONCRETE
OPEN CHAMBER

4 TRIFURCATION D = 1.20 m / 0.90 m / 0.95 m

5 PENSTOCK D = 0.95 m L = 1013 m
STEEL ALTO 37
THICKNESS 6-9 mm

Engineers + Consultants

EXAMPLE 13

POWERHOUSE

Q = 2.6 m3/s

EXAMPLE OF A POWERHOUSE WITH Q = 2.6 m3/s

An example of a powerhouse for a discharge of 2.6 m^3/s and an installed capacity of 1130 kW is shown on Drawings No. 40093-13-1 / 40093-13-2.

A Francis turbine has been selected for this scheme.

The lay-out illustrates the main basic elements required for small and mini hydropower plants.

Sufficient space has been allowed 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 to the corresponding requirements.

The power house should always be sited in a protected area near the river, and the tailrace must be placed high enough above the level of a 1000-year flood.

Access to the powerhouse should be secured all the year round.

The foundation slab should be made with PC 300 reinforced concrete. Stability of the slab must be checked by considering the maximum pressure at the butterfly valve during shut-down, and checking the pressure allowable on the natural ground.

The slab serves to support the equipment as well as providing a mass to dampen any possible vibrations during operation.

The powerhouse structure must be such as to effectively protect the equipment and instruments from the outside conditions.

The structure can be built with concrete or masonry blocks walls (min. 30 cm thick); if the walls are made of with natural stone masonry, the thickness should be of at least 50 cm.

The crane hook must be high enough for erection and maintenance work,in accordance with the indications of the equipment suppliers.

The roof should consist of transverse steel beams with a steel plate for bearing, to allow for the suspension of the crane beam.

The roof structure can be made of wood or steel (with sufficient strengthening for wind action).

The roof covering can be made of wooden planks ceiling, fiberglass insulation or corrugated metal sheets.

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

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

The powerhouse 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 gates which control the overflow have to be designed according to the suppliers specification.

Slots should be provided in the concrete walls to allow stop logs installation.

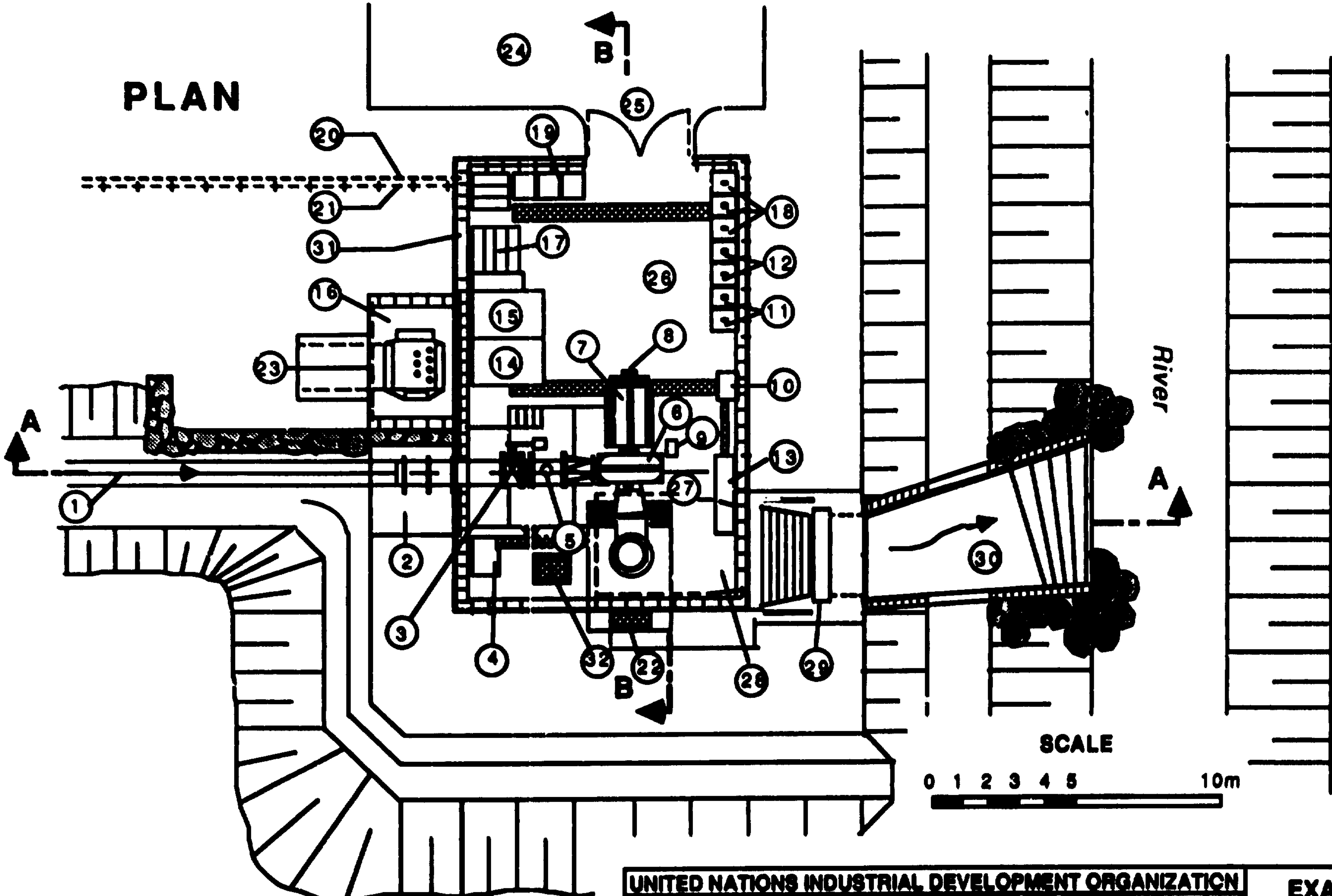
The connection between the tail race channel and the river must be solidly founded and should be protected, if necessary, with rip-rap.

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. CONTROL BOARD
12. EXCITATION CONTROL
13. MCC
14. GENERATOR SWITCHBOARD
15. SERVICE TRANSFORMER
16. MAIN TRANSFORMER
17. BATTERIES
18. LIGHTING AND DC DISTRIBUTION BOARD
19. REMOTE CONTROL OF WATERWAY
20. CABLE FOR LOW TENSION
21. CABLE FOR REMOTE CONTROL
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. DRAINAGE SUMP
33. LEAN CONCRETE PC 200

34. REINFORCED CONCRETE PC 300
35. STEEL BEAMS
36. CRANE 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

PLAN



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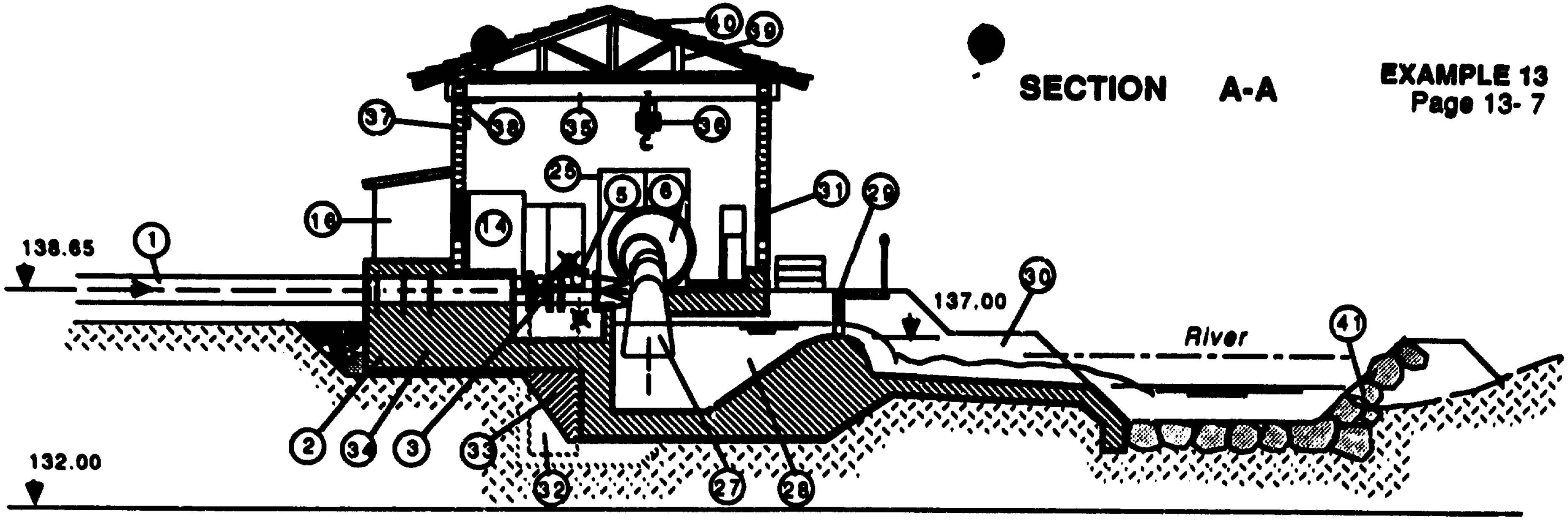
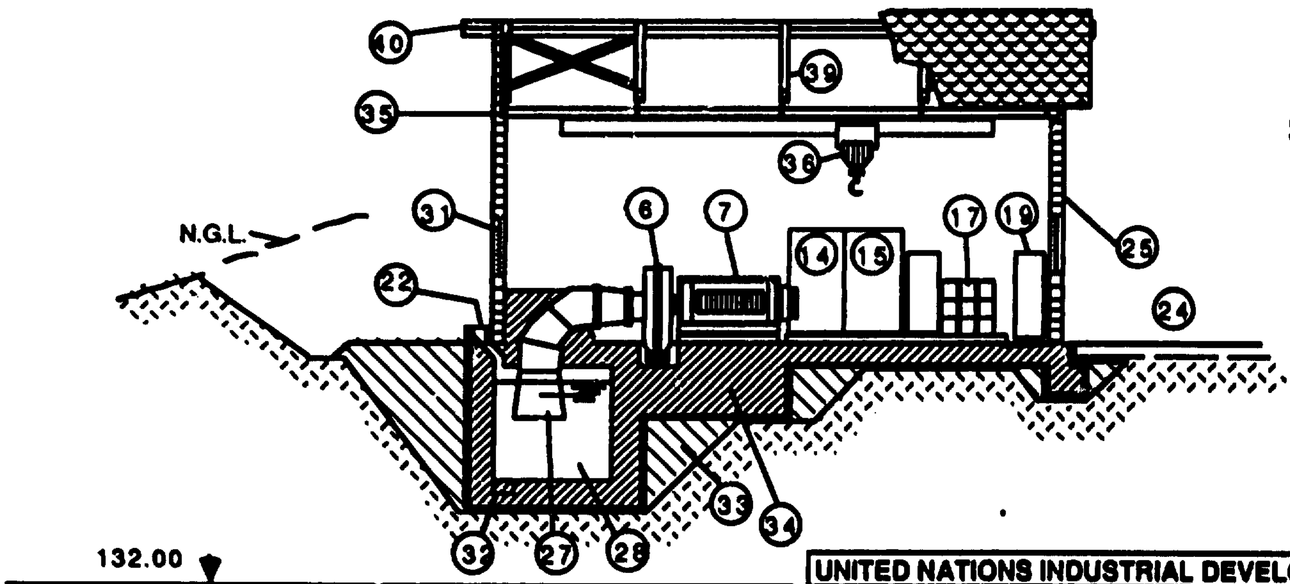
SECTION A-A



SECTION B-B



SCALE

0 1 2 3 4 5 10m

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