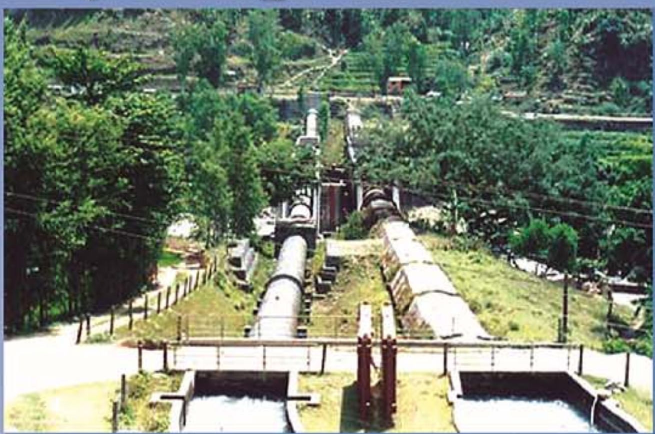
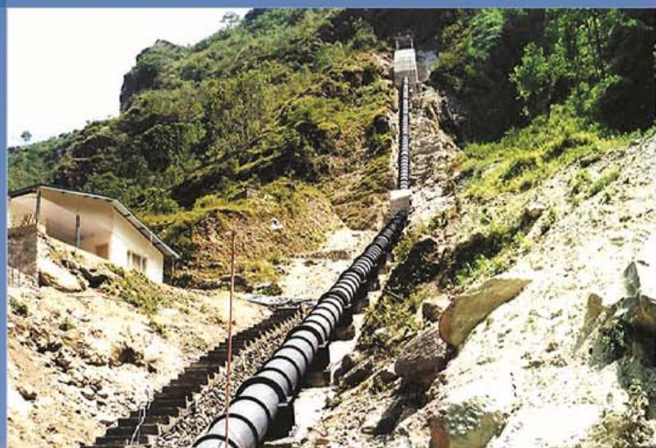
Design Guidelines for Water Conveyance System of Hydropower Projects



Outlet of Aqueduct-II, Trishuli HPP

Closed Conduit and Open Canal, Chilime HPP

Open Power Canal, Sunkoshi HPP



Penstock Structure, Chaku Khola HPP

Government of Nepal

Ministry of Water Resources

Department of Electricity Development

Foreword

Hydropower projects of diverse capacities occupy a prominent position in the process of infrastructure building of the country. For this reason, the hydropower projects need to be cost effective, sustainable, and environmentally and socially sound. Hence, the design of each component of a hydropower project demands meticulous planning, optimization and a judicious selection of the relevant design parameters, particularly in view of country's topography, young geology and nature of highly sediment laden monsoon rivers. In this regard, the Department of Electricity Development, in the course of examining the numerous project reports submitted to it for the issuance of licenses for survey or development in the areas of generation, transmission or distribution of electricity, has felt that there is a need to maintain a uniform standard and streamline the quality of the study reports in a manner consistent with international standards and norms, and compatible with the country's existing environment.

In this context, the Department has undertaken the task of formulating guidelines for the various components of hydropower schemes. This work, Water Conveyance System Design Guidelines, is one of such outputs that draws upon our past experience with other projects as well as international practices and relevant literature in the area. It is hoped that the Guidelines will serve as a useful reference material for practicing engineers and technicians as well as project proponents and developers.

The Department acknowledges with deep gratitude the invaluable suggestions, comments and the endeavor of many individuals, stakeholders from governmental and non-governmental institutions, and participants of the Workshop on Water Conveyance System in Hydropower Projects. We also extend our thanks to the Joint Venture Consultants, Hydro-Engineering Service (P) Ltd., Nepalconsult (P) Ltd. and GEOCE Consultants (P) Ltd. for undertaking the preparation of these Guidelines. Any suggestions / comments for further improvement and refinement in these Guidelines will be greatly welcome.

Jaya Keshar Mackay
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August 2006

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DEFINITION OF TERMS USED IN THE GUIDELINES

The definitions of terms used in these guidelines are as follows:

Anchor Block (anchor, anchorage):
It is a concrete block on a hillside for supporting and fixing the penstock.

Availability:
The availability is measured as the percentage of time available. It could be for water flowing in a stream, or for power production in a power plant. For hydropower the availability of power production depends on availability of water in a stream.

Aqueduct:
It is a channel for conveying water across a valley, river / drainage, canal, road or railway.

Banded Penstock (hoped penstock):
It is a pressure pipe reinforced by external ring-like or spiral like bandages to resist extremely high pressures.

Bifurcation:
Division into two branches.

Buried Penstock (covered penstock):
It is a pressure pipe laid in a ditch and subsequently covered.

Conduit:
A canal, pipeline or tunnel used for the conveyance of water.

Control Valve (conduit / penstock valve):
It is the valve, generally of butterfly type, installed between the surge tank or the headpond and the penstock or the pressure shaft, at the top of the latter.

Critical Head:
The hydraulic head at which the full gate output of the turbine equals the generator rated capacity.

Dam:
A structure for impounding water or raising the level of water.

Design Discharge:
The rate of flow designed for (i) water conveyance system, or for (ii) spillway

Design Head:
The head at which the runner of a turbine will operate to give the best overall efficiency under various operating conditions.

Dewatering:
Removing or draining water from an enclosure or structure.

Discharge:
The rate of water flow, generally measured in m^3/sec.

Drainage Area:
The area of land draining to a stream. It is also called **catchment area**.

Exposed Penstock (free penstock):
It is a pipeline located over the terrain or laid in an open ditch or led in a service tunnel.

Fixed Penstock (rigid penstock):
It is the term used for the entire pipeline if no displacement of its sections is restricted.

Flow-duration Curve:
A curve of flow values plotted in descending order of magnitude against time intervals, usually in percentages of a specified period.

Forebay:
The impoundment immediately above a penstock pipe line.

Full-gate Discharge:
The discharge through a turbine when the turbine wicket gates are wide open.

Head:
The difference in elevation between the head water surface above and the tailwater surface below a hydropower plant, normally measured in feet or metres.

Head Loss:
Reduction in generating head due to friction in the water passage to the turbine. It includes losses in trash rack, intake, penstock and in the conveyance length.

Headrace:
Water Conveyance conducting water to a hydropower plant.

Intake:
A structure to divert water into a conduit leading to the hydropower plant.

Load Rejection:
A fault condition that rapidly decreases the electrical load on the generating unit to no load.

Manifold (header):
It is the lowest portion of the penstock wherefrom the unit penstock bifurcate.

Maximum Surge:
It is the greatest amplitude occurring during surge oscillation, i.e. the maximum among the subsequent surges.

Net Head:
The gross head less all hydraulic losses except those chargeable to turbine.

Outage:
The period during which a generating unit, transmission line, or other facility is out of service.

Peak Demand Months:
The month or months of highest power demand.

Penstock (pressure pipe):
It is a pressure pipeline conveying the water in high-head developments from the head pond or the surge tank to the powerhouse.

Penstock Course (pipe course):
They are cylindrical or segment shaped pieces to be assembled into penstock sections.

Penstock Tangent:
It is a portion of the penstock between two subsequent break points. One tangent is generally anchored by one block at the lower end, long ones may be anchored at more than one point.

Rated Head:
The head at which a turbine at a rated speed will deliver rated capacity at specified gate opening and efficiency. However, for planning and design purposes rated head is identical to critical head.

Run-of-river:
A type of hydropower project that releases water at the same rate as the natural flow of the river (outflow equals to inflow).

Semi-fixed Penstock (semi-grid penstock):
It is the term used for the entire pipeline, consisting of tangents connected by flexible and / or expansible joints, if its displacement is restricted to a certain extent by friction over the supports located between anchorages.

Settling Basin:
A chamber designed to remove sediment from water by providing quiescent conditions that allow sediment to fall to the floor or the chamber. They are used in cases where sediment would otherwise block waterways or damage the turbine.

Spillway:
An outlet from a reservoir or section of a dam or on a side of channel designed to release surplus water that is not discharged through a structure or turbine or other outlet works.

Spiral Case:
A steel-lined conduit connected to the penstock or intake conduit that evenly distributes water flow to the turbine runner.

Stilling Basin:
The area on the downstream side of a spillway where water velocity is reduced to prevent erosion damage to hydraulic structures or the natural riverbed and banks.

Surge Tank:
A hydraulic structure designed to control pressure and flow fluctuations in a penstock or tunnel. It functions as a reservoir that temporarily stores or releases water to the turbine.

Tailrace:
A water conduit for conducting water away from a hydropower plant after it has passed through it, sometimes called an **afterbay**.

Thick-shell Penstock:
It is characterized by a ratio of the inner diameter and of the shell thickness, less than 20.

Transient:
That period during which events are changing with time.

Trashrack:
A rack or screen of parallel bars installed to prevent debris from entering the turbine.

Unit Penstock:
It is the pipe supplying a single turbine from the main penstock or the manifold.

Water Hammer:
Pressure changes in a pressure conduit or penstock that are caused by the flow variation with time.

Water Passage:
Conduits that convey water to and from the turbine runner. They include the scroll case, distributor and draft tube.

WR^2:
This is a constant also called “flywheel effect” that describes the rotating inertia of a given hydroelectric plant. In general WR^2 acts to improve the governing process. Usually, 90% of the mechanical rotating inertia is in the generator.

ACRONYMS AND ABBREVIATIONS

ACI	American Concrete Institute
ADB	Asian Development Bank
ANSI	American National Standard Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASNT	American Society of Non-Destructive Testing
ASTM	American Society of Testing and Material Standards for ANSI
AWS	American Welding Society
AWWA	American Water Works Association
B.S.	Bikram Sambat
BPC	Butwal Power Company
CPM	Critical Path Method
CSLIDE	Computer Analysis Program
CWI	Certified Welding Inspectors
DBC	Design Based Criteria
DERDAP	The Iron Gate
DMZ	Demilitarized Zone
DoED	Department of Electricity Development
EIA	Environmental Impact Assessment
EM	Environmental Management
EPA	Environmental Protection Act
EPC	Engineering Procurements Contract
EPR	Environmental Protection Regulation
EPRI	Electric Power Research Institute
ETL	Exact Transfer and Load
ETS	Educational Testing Service
EQ	Earth quake
FERC	Federal Energy Regulatory Committee
GLOF	Glacial Lake Outburst Flood
GPS	Global Positioning System
GRP	Glass/fibre Reinforced Pipe
HAD	High Aswan Dam
HDP	High Density Polythene Pipe
HES	Hydro-Engineering Services (P) Ltd.
HMG/N	His Majesty’s Government of Nepal
ICIMOD	International Centre of Integrated Mountain Development

IEE	Initial Environmental Examination
INPS	Integrated Nepal Power System
IUCN	International Union for Conservation of Nature
JICA	Japan International Cooperation Agency
JV	Joint Venture
KW	Kilowatt
Li	Lahmeyer International
MKE	Morrison Knudson Engineers
MW	Megawatt
NDE	Non Destructive Examination
NDT	Non Destructive Testing
NEA	Nepal Electricity Authority
NEPECON	Nepal Engineering Consultancy
NHE	Nepal Hydro and Electric Pvt. Ltd., Butwal, Nepal
PPA	Power Purchase Agreement
PQR	Procedure Qualification Reports
PVC	Polyvinyl Chloride
SMEC	Snowy Mountain Hydroelectric Company
SPFA	Steel Plate Fabricators Association
TEPSCO	Tokyo Electric Power Services Company
UNDP	United Nations Development Program
UPVC	Unplasticized Polyvinyl Chloride
US	United States
USBR	United States Bureau of Reclamation
USSR	The then Union of Soviet Socialist Republics
USACOE	United States Army Corps of Engineers
VOCs	Volatile Organic Compounds
VDC	Village Development Committee
WB	The World Bank
WECS	Water and Energy Commission Secretariat
WPS	Welding Procedure Specification

CHAPTER – I

INTRODUCTION

1.0 INTRODUCTION

1.1 Background

The adoption of policy by the then His Majesty's Government of Nepal to facilitate the private developer(s) interested to be engaged in hydropower development has played a positive role in attracting the private sector investment specially in the simple run-of-the-river derivation type hydropower projects. For achieving competitiveness, the private sector prefers to engage locally available professional capability to the extent possible. With a view to help the private sector in these endeavors, the Department of Electricity Development (DoED) has, among other things, realized the need for a set of Design Guidelines for Water Conveyance System.

1.2 Organization of the Guidelines

The guidelines have been organized into six chapters as follows:

Chapter-I	: Introduction
Chapter-II	: Investigation and Hydraulic Models
Chapter-III	: Design Guidelines
Chapter-IV	: Guidelines for Constructions
Chapter-V	: A Guide for Operation and maintenance, and
Chapter-VI	: Conclusion and Recommendations

Chapter-I contains study back ground, organization and limitations of the guidelines. Chapter-II covers the requirements of the investigations and surveys and the needs of hydraulic modeling in the design. Chapter-III is the main guidelines for the hydraulic as well as structural designs of the water conveyance systems. Chapter-IV describes the procedures to be followed during the construction of the water conveyance systems. Chapter-V outlines the works to be performed for smooth operation and maintenance of each of components of conveyance system. This chapter also contains a sample format for a periodic maintenance schedule. Chapter-VI draws the conclusions and presents recommendations.

1.3 Limitations of the Guidelines

This Design Guidelines for Water Conveyance System of Hydropower Projects is based on site specific data/ information, study of planned and implemented hydropower projects of Nepal and of typical hydropower projects constructed abroad and review of available manuals, text books, hand books and codes.

This design guidelines has been primarily intended as a source of ideas for engineers and technicians designing hydropower projects. The study is limited to the structural components that are generally designed as parts of water conveyance system of hydropower projects.

This document can be used to guide and eventually regulate the design quality of water conveyance system for hydropower development in Nepal. In order to have reasonable control of design practices there must be adequate guidelines which are to be accomplished by procedures and specifications. The guidelines describe the methods for providing a product or design output in this context. Design procedures usually establish the sequence of processes to achieve a required output while a system approach could consist of a network of procedures.

Specifications should also be part of guidelines, as they determine the size and quality or uniformity of the product or design output. Specifications are analogous to standard and criteria, and detailed specifications often overlap with guidelines. For example, conveyance structures are constructed by different methods depending on their specifications, but guidelines for their hydraulic analysis and design will be the same regardless of the construction mode.

Overall guidelines on the application of design principles will improve the consistency and enable to standardize the design effort of different agencies and consultants working in this sector. However, all aspects of conveyance system design cannot be completely standardized and produced in this document. Each hydropower plant is unique in nature and there may be diverse field conditions and different types of equipment availability which preclude recommendations that are wholly satisfactory and final. Bearing this in mind, designer should be cautious enough to use the document in an absolute term. Those particular works present unique technical problems with design and specialized knowledge and experience are often necessary. Design inputs for this category of works will have to come from outside sources.

This document is a first attempt for Nepal and constitutes simple a guide for design of water conveyance system for hydropower projects. Hence, it should not be considered as a full fledged handbook, manual or standards / codes for hydropower development. Any suggestion(s) from the users of the guidelines will be useful for further improving the guidelines.

CHAPTER – II

INVESTIGATIONS AND HYDRAULIC MODELS

2.0 INVESTIGATIONS AND HYDRAULIC MODELS

2.1 Investigations and Surveys Required for Design

The requirements of surveys, mapping and investigations pertinent to water conveyance system for hydropower projects are generally concerned with the following disciplines:

(i) Topography,

(ii) Hydrology,

(iii) Geology,

(iv) General Country's Condition, and

(v) Environment

2.1.1 Topographic Surveys

2.1.1.1 General

The procedures and standards to be followed in carrying out the topographic surveys required during the planning, design and construction of the Water Conveyance Systems for hydropower plants vary in different stages of system development. This sub-section highlights the procedures and requirements for carrying out the following surveys:

- The preparation of topographic maps, including benchmark establishment, survey traverses, spot leveling and mapping;
- The preparation of strip topographic maps of open channel & penstock lines and cross-sections; and
- Structure site surveys.

2.1.1.2 Survey Procedures

The following sub-section provides requirement of the existing data that should be collected, the procedures to be followed at identification, pre-feasibility, feasibility and design stages and finally, a summary of the survey standards.

a. Available Data: 1:25,000, 1:50,000 or 1:63,360 scale topographical maps and aerial photographs (may be at 1:200,000 scale and larger) covering the project area from DoED or Survey Department.

b. Identification: No topographical survey required if used available maps, photographs and information.

c. Pre-feasibility: At this stage, preliminary survey work is to be carried out using existing maps and photos to identify water conveyance alignments and structure locations. GPS could be employed at this stage.

d. Feasibility: At this stage detailed survey work is to be carried out. Based on this work, system layout is fixed and channels and different structures are designed. Accuracy in the survey work is thus of considerable importance. A successful functioning of the whole system is dependent on construction at the location and level. Establishment of bench marks and triangulation survey should be done at this stage.

e. Detailed Design and Construction: At this stage also need often arises for further survey of conveyance lines or structural sites, although most survey work is completed at the feasibility level. This may be due to alignment revisions or more minor structures have not been addressed. In addition, the system needs to be set out on the ground for land acquisition.

2.1.1.3 Types of Surveying

For carrying out the detail study of a hydropower project, it is necessary to prepare more accurate topographic maps of the project area. Following types of surveys are generally employed for meeting this objective:

Levelling: Levelling are done for fixing new bench marks, for checking the elevation of given location(s), for checking the accuracy of existing map(s) and for the control point survey.

Traverse Survey: Traverse surveys are carried out for the preparation of topographical maps of a given scale.

Control Points Survey: The control points survey is carried out by means of traverse survey for the purpose of establishing the base points for all necessary topographic survey works.

Plane Table Survey: The plane table surveys are carried out for preparing the larger scaled topographical maps. The survey works include the minor control point survey for horizontal and vertical control and the successive plane table survey. In the present day context, computer aided TOTAL STATION survey is also employed as a substitute to plane table survey.

2.1.1.4 Methodology and Data Processing

For fixing the elevation of each bench mark, survey by the direct levelling from the existing bench mark with known level is to be carried out and its accuracy is to be confirmed through the indirect levelling (Trigonometry) from the triangulation stations of which data are obtained from the Department of Survey.

For control point surveys traverse survey is employed. All the bench marks established need to be connected with the traverse route so as to be used as control points. The traverse route is generally arranged in a way to connect the existing triangulation stations of the Survey Department.

In the plane table survey, attention was paid to plot the change of topography as accurately as possible. Thus, the survey procedure, in which ridges, gullies or sharp changes of slope are checked and plotted firstly, need to be employed. The horizontal and vertical control of the plane table survey is strictly made by referring to the minor control points set prior to the plane table survey.

The traverse surveys are to be carried out using an electro-optical distance meter (EDM) for distance measurement and two transits of 1” reading for angle measurement. The coordinate of each control point is to be worked out based on the surveyed distance and the angle, the coordinates of the Survey Department Triangulation Stations and the Azimuth measured by astronomic observation. The computer aided TOTAL STATION equipment is also presently in common use for topographic surveying in hydropower projects.

For data processing different soft wares are used, the conventional method of data processing has gradually become out dated.

2.1.1.5 Accuracy

Accuracy in Conducting Triangulation: The suggested accuracy required for different order/ class of triangulation surveys are as given in the following table[1].

Class of Triangulation	Average Length of the Side (km)	Accuracy in Measurement of the Angle (sec)
I	20 – 25	+/- 0.7
II	13	+/- 1.0
III	8	+/- 1.5
IV	4	+/- 2.0

For hydro-energetic surveys, more denser sets of triangles with side lengths of 1 to 3 km or even micro-triangulation with side lengths of less than 1 km may have to be created.

Accuracy for the Control Point Survey: The traverse employed for control point survey needs to be closed with a satisfactory accuracy, satisfying sufficiently the following standard[2] required for the control point survey:

Coordinate: Within 1/5,000 and 10.0 cm + 20.0 cm $\sqrt{S}$
(S is survey distance in km)

Elevation: Within 10.0 cm + 3.0 cm $\sqrt{n}$
(n: number of points surveyed)

2.1.1.6 Mapping Standards

The following topographic standards (STD) for mapping are generally recommended:

STD	Principal / Use	Details
1.	Desk Studies	Use existing maps (scale 1:25,000, 1:50,000 and / or 1:63,360), aerial photos and satellite imagery
2.	Identification	Use existing maps and aerial photos but supplement with site inspection.
3.	Pre-feasibility	Use existing maps and aerial photos for reference purposes. Measure distances between key features located on photos to fix photo scale. Prepare tentative system layout using photo enlargements as base map. Indicate proposed system alignments and structures.
4.	Feasibility Study	Use existing aerial photos with site measurement of key features to support design. Take spot levels on 100 m grid to produce 1:2,500 scale topo maps with contour intervals of 0.5 to 1.0 m and carry out site surveys of major structures at a required scale.
5.	Detailed Design	Set out conveyance system and cross-drain layout in the field. Carry out line and structure surveys for revised alignments or locations or works not already surveyed.
6.	Construction	Use design drawings, check and agree levels and dimensions before and after construction for measurement purposes. Carry out additional site surveys if required.

Other details in this respect are provided in the Guidelines for Study of Hydropower Projects, published by Department of Electricity Development (DoED) in December, 2003, Kathmandu.

[1] *Source: Hydro-Energetic Survey by Professor Dr. E. V. Bliznyak and others*
[2] *Source: Feasibility Report on Sapta Gandaki Power Development Project by JICA, 1983*

2.1.2 Hydrological Surveys

The most important type of hydrological data required for the design of water conveyance system in a hydropower project is the long-term stream flow record that represents the flow available for power production. Hence, the first step in the study of hydrology is to collect the long-term historical data of the river under study.

If no previous records of flow exist of the river to be developed, or from a catchment close by, the alternative approach is to establish a short-term river gauging station at the proposed site in the river and observe the daily water level readings. Besides, discharge measurements are carried out from time to time to cover as far as possible the different levels of water so that a rating curve could be developed. This will help to compute the daily discharge data.

This short-term records will be related to long-term observations of flow or precipitation from a site within the same hydrological region and generate the long-term records.

Based on these long-term flow data a flow duration curve is developed. This curve is used to summarize stream flow characteristics and can be constructed from daily or monthly stream flow data. The curve shows the percentage of time that flow equals or exceeds various values.
The decision makers will decide at which probability of exceedance the project will be designed. The design flow will be determined accordingly from the flow duration curve. Based on this design flow the conveyance system will be designed.

Apart from the estimation of this design flow, the study on hydrology in the design of conveyance system is also concerned with the hydrology of the cross-drainages that encounter in the conveyance system particularly in the canal system.

In the design of the cross-drainage structures, the estimation of the high flood (design flood) is the must. This study is done either by the statistical analysis or by the use of empirical methods or rational formulae. For the streams or rivulets where the long term hydrological data are available, the best approach for the estimation of design flood is the statistical analysis. In this method the flood frequency analysis is carried out by using the statistical distributions. The most commonly used distributions in Nepal are log-normal distribution, log Pearson type-III distribution and Gumbel distribution. The generalized equation of these distributions is given by the formula:

$$Q_T = \overline{Q} + K_T * S \quad \text{---} \quad (2.1)$$

Where, Q_T = Flood discharge (m^3/sec) for T-years return period,

$\overline{Q}$ = Mean of flood series (m^3/sec),

K_T = Frequency factor for T-years return period, and

S = Standard deviation of the flood series.

For the first two distributions mentioned above, the first step is to convert the flood data into logarithmic series. Usually, the data are converted into natural logarithm (ln) for the log-normal distribution and into logarithm to base 10 (log) for the log-Pearson type-III distribution. The frequency factor (K_T) for the type of distribution is available in any hydrology text books.

In the context of Nepal it is of little chance that the historical long term data for the small streams or rivulets are available. So in such case the rational method is widely used. This method is based on the principle of the relationship between the rainfall and runoff. It also takes care of the catchment characteristics. It is expressed by the formula:

$$Q_T = CiA/3.6 \quad \text{---} \quad (2.2)$$

Where, Q_T = Flood discharge (m^3/sec) for T-years return period,
i = Intensity of rainfall (mm/hr) for the duration at least equal to the time of concentration, and
A = Catchment area in km^2.

The time of concentration (t_c) is calculated by the relation:

$$t_c = 0.000324*(L^{0.77}/S^{0.385}) \quad \text{----------------------------} \quad (2.3)$$

Where, L = Length of drainage basin in meter measured along the river channel up to the farthest point on the periphery of the basin from the point under consideration, and

S = Average slope of the basin from the farthest point to the point under consideration

As far as sediment is concerned, size distribution of the sediment data and their hardness rather than the total yield are more important for design of the desanding basin, because the harder particle and larger size passing through the turbine can wear and erode various machine parts. Hence, petrographic analysis as well as the determination of hardness of sediment particles is essential.

2.1.3 Geological and Geotechnical Investigations

2.1.3.1 Investigation Procedure

The geological and geotechnical investigation adopted at different stage of study – site identification, pre-feasibility study and feasibility study for hydropower development delivers certain amount of information useful for the design purpose. The information obtained in the initial stage of investigation provides usually the information of general nature. Information of the subsequent phase will be more specific and qualitative. Review of the findings obtained in each stage should be considered as a basis for planning the geological / geotechnical investigation works for each subsequent stage. At design stage geological / geotechnical investigation should be carried out to fill up the gap that is not covered at the feasibility stage or in other proceeding stages. The geological / geotechnical investigation pertaining to the study of the structural components of the water conveyance system involves examination at the following locations:

- Desanding basin;
- Canal;
- Pipe;
- Tunnel;
- Cross drainage (Aqueduct, Siphon and Super-passage)
- Forebay / Surge tank / Surge shaft;
- Penstock; and
- Tailrace

The study should be undertaken to achieve the followings:

- Preparation of engineering geological map covering the desanding basin, canal alignment, forebay, penstock, powerhouse and tailrace.
- Preparation of engineering geological map and section covering the tunnel alignment and its portals, and surge tank / surge shaft locations including derivation of rock quality and overburden cover in the tunnel route.
- Preparation of engineering geological map of the cross drainages covering the locations of the aqueduct, siphon, super-passage, flume and overflow spillway.
- Sampling and testing for physical and mechanical properties of soil and rocks representing the desanding basin, canal alignment, tunnel route, tunnel portals, surge tank / surge shaft, forebay penstock and tailrace sites;
- Drilling and geological field testing at the desanding basin, tunnel portals, forebay surge tank / surge shaft, and penstock locations; and

- Sub-surface examination of the ground condition through conduct of 2D resistivity survey and seismic refraction survey.

Geological / Geotechnical investigation in the different stages pertinent to the examination of favourability of the water conveyance route for the hydropower development is to be carried out following the sequential steps given below.

Identification / Reconnaissance Study

- Review the available information relating to regional geology, photo geology and tectonics, and topographic base map covering the surroundings of the hydropower development project area for identification of the alternative water conveyance alignments.
- Define tentative water conveyance routes in the topographic base map.
- Conduct walk over survey around the locations of the alternative water conveyance routes in consideration of the ground condition and categorization of the options for the further study.
- Identify new potential water conveyance alignments in case those tentative water conveyance locations do not justify suitability in conjunction to the actual ground condition.
- Recommend the better conveyance locations in case that emerged after the field observation.

Pre-feasibility Study

- Perform review of the alternative water conveyance routes identified during the reconnaissance study and reconfirm the best option through examination of those locations in consideration of the favourable and sound ground condition.
- Carry out the traverse survey along the alternative conveyance routes for determination of the general geological, geo-morphological and stability characteristics of the alignments for subsequent comparison to establish the best option.
- Recommend (i) modification on the previously identified conveyance alignment if necessary and (ii) the better conveyance option in case that emerged after the reconfirmation field visit.

Feasibility Study

- Conduct inception / initial field visit so as to confirm that there is not better alternative water conveyance alignment than those identified in the pre-feasibility stage, and recommend if any new conveyance alignment is appeared.
- Undertake detailed geological / geotechnical investigation covering the best water conveyance option as supported by favourable terrain and geological conditions.

Detailed Design

- Perform additional field examination / investigation to gather gap on geological and geotechnical information required for detailed design purpose, and supplement findings by the laboratory testing.

Construction

- Undertake observation and recordings at the foundation excavation sites done during the construction period so as to obtain actual sub-surface geological information at the water conveyance route and sub-structure location.
- Suggest modification in the design works in reciprocation to the sub-surface geological changes (anticipated and unanticipated ones).

2.1.3.2 Geological / Geotechnical Investigation Standard

Accomplishments of the following activities are recommended while undertaking the geological / geotechnical investigations at different stages.

S. No.	Stages of Study	Activities to be undertaken
1.	**Site Identification or Preliminary Study**	
1.1		Presentation of regional geology and tectonics of the surrounding of the project area based on review of available literature, topographical maps (1:25,000 or 1:50,000 scale), geological map (1:50,000 or 1:63,000 scale), aerial photographs (1:50,000 or 1:63,000 scale).
1.2		Identification of the potential locations of the water conveyance route and respective structure sites in the field by conducting walk over survey.
1.3		Comparison of the identified potential options on the basis of favourability in reference to terrain and geological conditions.
2.	**Pre-feasibility Study**	
2.1		Deliverance of the review report based on the site identification report and other available maps, geological maps and sections and aerial photographs and landsat images.
2.2		Generation of overview perception on regional geology and geomorphology of the identified sites with supplementation of the respective map and sections.
2.3		Conduct of field survey at those potential sites and at new better locations covering the respective water conveyance route and structure locations for confirmation of the best options in terms of (i) favourability of the geological and stability characteristics, and (ii) suitability of conveyance route either as canal option or as tunnel option or as combination of both options.
2.4		Derivation of the findings covering the tentative layout plan indicated on a base map in scale 1:5000 or 1:10,000 prepared by enlargement of the existing topographic map and aerial photographs.
3.	**Feasibility Study**	
3.1		Presentation of the review report based on the pre-feasibility report and other relevant literature, topographical maps, geological maps and section, and aerial photographs depicting (i) regional geology and tectonics of the region around the project area, and (ii) geological, geo-morphological and stability characteristics of the proposed water conveyance system and structure locations.
3.2		Accomplishment of an initial field survey for reconfirmation of the best option indicated in the pre-feasibility stage and recommendation if there is any additional better option.
3.3		Conduct of detailed geological and geotechnical studies covering (i) sufficient ground of the desanding basin, forebay and powerhouse locations as essential for the construction purpose, (ii) 100 m wide corridor along the canal alignment including dissecting cross-drainages and penstock line and (iii) the area bounded by the contours of same level as the tunnel route observed on the ground surface and the corresponding alignment.

S. No.	Stages of Study	Activities to be undertaken
3.4		Presentation of the engineering geological maps of the desanding basin, forebay, penstock line, powerhouse and tailrace locations at a scale 1:1000 depicting overburden thickness and characteristics of the soil and rock exposure occurring at those locations. The map shall contain geological sections at a scale 1:200 or 1:400 as appropriate for the design purpose.
3.5		Delivery of the engineering geological map for the canal route and cross-drainages at a scale 1:5000 with illustration of characteristics and distribution of the soil and rock exposures including the overburden thickness. Such engineering geological map shall reflect the characteristics of the soil and rock present at the aqueduct, siphon and super-passage locations.
3.6		Presentation of Engineering geological map prepared covering the tunnel alignment and its portals and the possible adit locations at a scale 1:10,000 or 1:20,000 should reflect the rock type and discontinuities, quality of rock and location of the rock exposures. Supplementary geological section prepared along the tunnel alignment should illustrate (i) the overburden thickness, (ii) the rock type to be encountered and the likely Rock Mass Rating and Q-values to be observed, and (iii) the corresponding tunnel support to be required.
3.7		Deliverance of a report including maps and sections in an appropriate scale for design requirement reflecting the finding of the field investigation and laboratory testing.
4.	**Detailed Design**	
4.1		Presentation of maps and sections prepared in the Feasibility Study Report in an appropriate scale required for design purpose.
4.2		Identification of the data gap in the Feasibility Study stage for further investigation.
4.3		Augmentation of the data gap after completion of additional geological / geotechnical investigation.
5.	**Construction**	
5.1		Examination of foundation excavation locations done for construction purpose.
5.2		Determination of changes on anticipated and unanticipated ground condition at the construction site.
5.3		Recommendation of remediation measure in accordance with the changes in ground condition on construction works.

2.1.3.3 Geological / Geotechnical Investigations Required for Different Structural Components of Water Conveyance System

This section defines the requirement of geological and geotechnical investigation at the different structural components of the water conveyance system for the design purpose.

Desanding Basin

Sufficient open and flat ground having stable side slope condition and situated a few meter higher than the current river flow level should be identified for the location of a desanding basin. The best location for this purpose will be the old terrace deposit present in the bank of the river. Surface examination at such location should be conducted to derive the (i) distribution, thickness and characteristics of the overburden material, (ii) distribution and quality of the rock exposures, and (iii) slope and stability conditions in the uphill side. Test pitting or drilling as appropriate supplemented by the seismic refraction survey or 2D resistivity survey is to be undertaken not only to establish the nature of the geological materials present up to influencing depth of the foundation but also to ensure that there is not any unfavourable sub-surface condition for the foundation. The engineering geological map at a scale 1:1000 with supplementation of the section, drilling log, and result of the seismic refraction survey or 2D resistivity survey including regional geology should be delivered in the useful form for the design purpose. Prior to performing the detailed investigation in such location, the observed ground level should be in conformity with the level of the corresponding forebay or surge tank or surge shaft.

Canal

The soil and rock materials as well as the slope and stability conditions existent along the canal is the primary controlling factor in the selection of the canal option in place of pipes or tunnel for conveying water in the hydropower development. Despite high prospect of hydropower canal in the hills and middle mountain physiographic regions of the country, possibility for the large dimension canal is limited due to steepness and fragile geological condition of the terrains along the side slopes of the river valley. Such conditions are usually encountered in the middle part of the canal section that is away from the desanding basin, away from the forebay and away from the tunnel portal locations.

The investigation findings obtained for canal through observation, pitting, auger boring, sampling and testing should demonstrate the distribution and characteristics of soils and rocks and the favourability of the terrain condition for excavation and construction which is to be depicted in a map of scale 1:2,000 or 1:5,000 and in a section at scale 1:200. Examination outcome should also reflect the information about the materials to be encountered at the foundation such as air slaking materials, low density materials and expansive soils.

Pipes

The terrain condition indicating unfavourability for excavation of canal may in some case exhibit the possibility of the pipe option as the conveyance for hydropower development because it will require minimum excavation space for pipe installation. The geological and geotechnical study for the design of the pipe route should emphasis the distribution and characteristics of the rock and soils, thickness of the overburden soil, position of the unstable areas, and locations of foundation problem materials. The outcome of the investigation should be presented in a map of scale 1:2000 or 1:5,000. The investigation should also be oriented to define the areas of talus and scree materials, air slaking materials, low density materials and expansive soils.

Tunnel

The study for the tunnel option should be preferred in the following situation:

- Presence of high sloping terrain due to which surface water conveyance route is not possible;
- Poor stability condition in major section of the surface water conveyance option;
- Favourable terrain condition with formation of a loop in the river course resulting considerably minimum water conveyance length while considering the tunnel option in comparison to the canal or pipe option;
- Prevalence of appreciable head consideration of tunnel option in place of surface conveyance route;
- Slope condition of the ground is not suitable for the required large dimension canal.

The investigation should address the following main geotechnical issues which bear on excavation stability short and long term safety and cost.

- Rock type and its strength,
- Orientation of bedding and discontinuities,
- Geological structures and shear zones dissecting the tunnel route,
- Abrasivity of the rock material / rock mass,

- Weathering, water tightness and likely ground water problem,
- Characteristics of MBT and MCT if existent,
- Rock mass classification reflecting the Q-value and RMR-value,
- Overburden thickness and type of support required for the tunnel route.

Forebay

The location of a forebay corresponding to the canal or tunnel option will be appropriate where adequate large flat or gently sloping open ground is present. The site for the forebay location will be suitable in the either ridge top or spur top or mid-slope region. Examination of the ground surface at the forebay region should be oriented to establish the distribution and characteristics of the overburden soils and rock exposures as well as the stability situation in the surroundings. Supplementary drilling activity, 2D resistivity survey or seismic refraction survey are performed to deduce the sub-surface geological information regarding overburden soil / bedrock configuration characteristics.

The geological map (1:1000) and section should be prepared to reflect the position of the soils and rock exposures including their characteristics.

Surge Tank / Surge Shaft

The surge tank should be considered as a component of the conveyance system in absence of suitable ground for the forebay or surface penstock route. A surge tank is provided at the junction of the headrace tunnel and the penstock for the purpose of not only absorbing the water oscillation at small amplitudes under the normal condition but also intercepting the pressure wave to be generated due to water hammer action upon sudden closure of the turbine in case of load rejection. In consideration of above requirement the surge tank is preferred to be located at strong and stable rock condition as far as possible so that it can face the water hammer action during the project life. The sound and stable condition at its access tunnel is also to be taken into account while location for the surge tank is identified.

Penstock

The position of the penstock will be guided by the respective location of the forebay and powerhouse. As the penstock line follows the terrain towards the down slope direction between the forebay and powerhouse, the study of the soil and rock characteristics, overburden soil depth and the slope condition and its stability are the important factors that requires consideration while undertaking geological / geotechnical investigation. Surface examination along the penstock line should be supported by the drilling, 2D resistivity survey or seismic refraction survey in order to determine the position and characteristics of the overburden soils and the depth of bedrock.

The map and section to be presented at a scale 1:1000 should indicate characteristics of the soil and rocks, and their extension at depth.

Burial of the penstock may be necessary during construction to avoid high angle unstable slope cutting of the overburden soil. Such situation is encountered involving deep soil cutting.

Identification of sub-surface penstock (vertical shaft / inclined shaft) is required wherever surface ground condition is not suitable due to occurring of unfavourable slope condition that means presence of either steep slope or unstable slope. In case of selection of the inclined underground penstock, it can also be considered to build into smaller sections comprising of two or more inclined shafts connected by the horizontal tunnel so as to ensure save in the cost of pipe and construction time. This consideration will be practically possible provided the suitable site for work adit is available in the middle reach of the inclined shaft. Such arrangement is apparent in the penstock of the Kulekhani-II project. For this purpose, the rock condition along the penstock line should be competent and stable which should be assessable from the surface geological mapping and seismic refraction survey or 2D resistivity survey.

Tailrace

Surface examination done at the powerhouse site with supplementation of the sub-surface investigation carried out through the drilling, 2D resistivity survey or seismic refraction survey will provide the necessary information about the nature of overburden soils and rock occurred at the tailrace location as well. The findings of the geological / geotechnical investigations performed covering the powerhouse and tailrace locations should indicate the characteristics of the overburden soil and bedrocks in a map of scale 1:1000 and in a section of scale 1:200 or 1:400 as appropriate.

In case the powerhouse location is placed in the underground, the corresponding tailrace will be in the form of tunnel. Such tailrace tunnel should be driven through the sound and stable rock. In the context of the hydropower development in Nepal, the length of tailrace tunnel differs greatly in reference to the geological and topographical conditions of the project area. The longest tailrace tunnel in Nepal is one kilometer long for the Andhi Khola Hydropower Project.

Cross Drainages

Geological / geotechnical investigation at the different cross-drainages is to be conducted to determine the characteristics of the overburden soils and bedrocks, and their likely position at the foundation of either the aqueduct or the siphon or the super-passage or the overflow spillways or the flumes. Assessment of the foundation condition will be based on the surface examination of the overburden materials and rock exposures present in the river bed and on the banks. Sub-surface examination by pitting may be necessary in absence of any rock exposures. Geological section representing the respective structural component location should be prepared in a scale 1:200 or 1:400 as appropriate.

2.1.3.4 Requirement of Seismicity Study

General

The geology of Nepal has resulted from the collision between the Indian and Eurasian tectonic plates. The initial conveyance between the plates involved the closing of the ancient sea between the two land masses and the subduction beneath the Eurasian plate of the denser oceanic plate underlying the Indian Continental plate. Between 40 and 50 million years age, the gap between the continents was completely closed and continent to continent collision commenced. From this point on, the evolution of the present day Himalayan mountain chain began with sediments, igneous intrusions and the progressive elevation of land mass. Therefore, the Himalayan arch in which whole Nepal is included is one of the most seismically active areas in the world.

The location of the initial interaction and conveyance is thought to be the Indus-Tsangpo Suture (ITS). With further conveyance, the ancient sea which separated the land masses was closed, and the Continental Indian plate began to sub-duct beneath the Eurasian Plate. The active boundary between these tectonic plate, progressively shifted southwards from ITS, first to the main central thrust (MCT), and then to the main boundary thrust (MBT). All these structures are of continental dimensions and have been the location of great earthquakes. The southern line of the Siwaliks is bordered by the Himalayan Frontal Fault (HFF), which although comparable in length to MCT and MBT, is thought to be composed of a series of shorter segments.

Study Requirement

The study on seismicity will be aimed to derive the seismicity characteristic of the project areas and surroundings for the design purpose. Its ultimate objective will be to establish the seismic coefficient to be considered for designing the structural components of the water conveyance systems. The study should be emphasized on the following aspects covering 200 km of the project area.

- Review and interpretation of the available historical and instrumental seismicity data for defining significant earthquake in consideration of the factors such as the location, distance, magnitude and intensity.
- Review and interpretation of the geological maps, aerial photographs and landsat images for categorization fault for identification and categorization of the different faults and earthquake sources.

- Determination of reoccurrence period and magnitude for the significant earthquakes.
- Derivation of the peak ground acceleration for Maximum Design Earthquake with consideration of the seismic co-efficient values recommended in the region for the similar project.
- Risk assessment in consideration of ground movement dislocation and rock shattering of fault, ground creep, landslide and rock fault.

Seismic Design Parameters

The site accelerations due to an earthquake on the identified tectonic structures have to be assessed based on acceleration attenuation distance relationships derived from recent worldwide records in similar geological environments. It is considered that MBT is capable of generating a maximum credible earthquake (MCE) of magnitude 8 on the Richter scale. The current practice of the design of important water retaining structure is based on the 84th percentile peak horizontal acceleration resulting from the MCE. The large reservoirs are to be designed with an acceleration value of 0.5 g corresponding to a return period of 1000 years. Due to lesser sensitivity to catastrophic damage leading to loss of lives and property conveyance system structures are recommended to be designed with an acceleration value of 0.15 to 0.25 g risk and economic sensitivity has to be taken care off by the designer while adopting actual design value of the acceleration.

2.1.3.5 Safety Consideration for Geological and Geotechnical Investigations

Conduct of all geological and geotechnical activities in a safe manner can prevent injuries to persons and damage to properties and equipment. All injuries can be prevented. No tasks shall be undertaken until adequate safeguards have been provided to protect every involved professional from injuries due to recognized hazards. All professional involved in the above mentioned activities had the responsibility for safety of themselves and others. Where it is not reasonable or practical to eliminate the sources of danger, measures must be taken to use safe procedures, guards, safety devices and protective clothing. It is also important that the professionals take an active and personal interest both in own safety and in the safety of his work mates.

Safety Code of Conduct

- Observe all safety rules;
- Wear personal protective equipment where required;
- Be enthusiastic about safety;
- Give safety priority among the problems;
- Understand the job safety analysis or/ and equipment procedures of each job;
- Take time to evaluate possible hazards before beginning any job;
- Correct unsafe acts immediately even if it requires stopping the job; and
- Be alert for unsafe conditions.

General Safety Rules

A safety rule, for the most part, is a common sense applied to working habits. It is impossible to list a rule covering every situation, but below are safety rules which apply to all professionals involved in geological and geotechnical investigations at all times.

- Approved safety hammer, safety glasses and safety foot wear must be worn while undertaking investigation on the ground surface, and a safety hat is mandatory in the drilling job either on the surface or under ground;
- Additional safety hat, safety lamp and safety cloths must be used while performing the investigation under ground;
- Dark glasses should not be worn in the under ground.

It is the responsibility of operator to eliminate the risk for a accident with the instrument in the 2D Electrical Resistivity Survey. The steps mentioned below should be strictly followed to avoid accident.

- Measurement should never be taken during a thunderstorm;
- Disconnect the cables if a thunderstorm came up while undertaking measurement;
- Avoid touching of the bare cables and connectors by the person and animals;
- Operate the instrument by the authorized personnel only; and
- Keep an eye always on the cables and instrument.

2.1.4 Considerations of General Country's Conditions

During design of any structural component, in addition to the site specific conditions made known by site specific surveys and investigations as described in the above sections, the following general conditions prevailing in the country should also be considered.

2.1.4.1 Terrain Conditions

In the Terai plain the prospects of hydropower development is quite low. The terrain conditions of mountainous regions of the country are favourable for tunnels. The prospects for open canals are limited due to steep terrain conditions. Stability is of the major concern. The largest hydropower canal so far built in the mountain terrain of Nepal is for the capacity of 45 m^3/sec at Trishuli. It was built with a combination of the open and covered canals, to avoid some unfavourable steep slopes along the canal route on the right bank, it had also to pass through left bank. As the result, two aqueducts crossings over main Trishuli river had to be constructed for following the favourable terrain conditions situated on both the banks.

2.1.4.2 Hydro-meteorological Conditions

The wide variation of climate from subtropical to alpine, mainly due to the large range of altitudes encountered in the country has influence on hydrology. Domination of south-east monsoon, not effective north-west or winter monsoon, existence of large number of glaciers, high altitude lakes including glacial lakes, low land Terai plain in the south-all they influence in hydrology. Hydrologically the rivers of Nepal can be divided into three following types:

- Purely rain-fed
- Rain plus snow-fed
- Rain plus snow plus glacial-fed

The small rivers (purely rain-fed type) flowing particularly in the Terai plain almost dries up in the dry season. But the small mountainous rivers roar up during cloud bursts. The damages due to cloud bursts and glacial lake outburst floods (GLOFs) are large. The examples are breakdown of penstock pipe of 60 MW Kulekhani-I hydropower project by debris flow occurred from cloudburst of 1993 monsoon and washing out of Namche hydropower project (600 kW nearly completed) in August 1985 due to outburst flood of Dig Tsho Glacial Lake.

The limited data available on sediment measurements show large variations. The highest concentration is during monsoon, in the time of cloud bursts and GLOFs. The monsoon is mainly responsible for surface erosion; hence sediment yield closely follows river discharge peaking, generally in August. The indirect evidence suggests that during GLOF, the sediment concentration is around 100,000 mg/l.

2.1.4.3 Geological Conditions

General

In broad sense, Nepal Himalayas have been divided into the five tectonic zones. The transverse and longitudinal faults are the other major structural elements present in the different tectonic zones. The thrusts and faults are considered to be still active in some locations. The tectonic zones as identified from the south to the north of the country are as follows.

- Terai zone
- Sub-Himalaya / Siwalik zone
- Lesser Himalaya zone
- Higher Himalaya zone
- Tibetan - Tethys zone

The Himalayan Frontal Thrust, Main Boundary Thrust and Main Central Thrust (Ganser, 1964 & Hagen, 1969) existing in the country exhibit linear continuity throughout the country in the east-west direction. Those tectonic show distinctive lithology, structural features, geological history and rock quality. As an illustration the rock of the Siwalik regions are soft, weak and less jointed whereas the rocks present in the Lesser Himalaya zone, Higher Himalaya zone and Tibetan - Tethys zone are generally medium hard to hard, medium strong to strong and usually jointed. Like the other parts of the Himalayan Range, the regions of Nepal are tectonically active and susceptible to deformation and frequent earthquake. As a result, the rocks present in those tectonic zones are found fragile, fractured and jointed giving rising to rocks of different quality the instabilities and landslides are comparatively more apparent on the ground surface in the Siwalik zone.

The following description provides the general geological and tectonic information of the different zones. Site investigation is necessary to determine the specific geological and structural features respective to the project ground condition.

Terai Zone

The Terai zone representing the southern tectonic division of Nepal, in true sense, occupies the northern most edge of the Indo-Gangetic alluvial plain of the Pleistocene to Recent age often demarcated to the north by an active fault called the Main Frontal Thrust or Himalayan Frontal Thrust. This zone comprising of clay, silt, sand and gravel is not much of significant in consideration of the design and construction of the conveyance system for hydropower development as the hydropower potential zones are virtually absent in its surroundings. However the Terai zone is likely to be the site for the powerhouse location either for run-of-river projects with conveyance system dissecting the Sub-Himalayan Zone or for high dam river type scheme with conveyance route traversing the Siwalik rocks.

Sub-Himalaya / Siwalik Zone

The Sub-Himalaya zone, widely known as the Siwalik or Churia hills, emerges as the southern most mountain range in the Himalayas. It is bounded on the north by the Main Boundary Thrust (MBT) and on the south by the Main Frontal Thrust. The Siwalik zone includes the fluvial rocks comprising of clay stone, mudstone, siltstone, sandstone, conglomerate and boulder beds of Neogene age. The Dun valleys occurring in between the Siwalik rocks include the Quaternary fluvial sediments. In general the Siwalik rocks dip northward at varying angles with a number of east-west trending thrusts.

The Siwalik zone is not much of important from the view of hydropower development due to absence of potentials in the river originated in this zone. However the 2400 m long tunnel works of approximately 3 sq. m dimension of the 1 Mw Tinau hydro-electric project is an illustration of the hydropower conveyance system in this zone. The project has used the water source derived from the Lesser Himalayan zone.

Existence of such characteristics is reported in the test adits driven for 672 m length in Karnali-Chisapani Multipurpose Project. Likewise the rock quality determined through examination of the 100 m long test adit dug for the Sapta Gandaki Hydro-electric Power Project is reported to be of Class-C with seismic velocity range of 1.7 km/sec to 3.6 km/sec. On the other hand, presence of the rock of adverse quality having nature of loosening and squeezing ground cannot be ruled out in this zone.

Lesser Himalaya Zone

The Lesser Himalaya Zone is bordered in the south by the Main Boundary Thrust (MBT) and in the north by the Main Central Thrust (MCT). The zone is comprised mainly of the sedimentary and meta-sedimentary rocks such as phyllite, slate, quartzite, limestone and dolomite ranging in age from Pre-Cambrian to Eocene. Some specific rocks like granite, granitic gneiss are also present in some locations. Presence of extensive superficial cover of the boulder, gravel and sand beds are the common association in the banks of the major rivers in this zone.

The Lesser Himalayan Zone is highly significant for hydropower development by reason of inherent hydropower potential, favourable geological condition and fairly good infra-structural situation. Almost all the existing hydropower projects are built in this zone in exception of the Tinau Hydropower Project.

In general, the rocks observed in the different hydropower project built in this zone are of good to poor nature corresponding to rock classes - B, C and D. Rock types of extreme quality such as exceptionally poor and exceptionally good rocks are also encountered. Their proportion varies in the different projects depending upon the rock type and their position in reference to the areas of extreme stress condition. Hence the future hydropower projects in this zone are likely to encounter different ground condition as indicated above.

Higher Himalayan Zone

The Higher Himalayan Zone includes approximately 10 km thick high grade metamorphic rocks that lie between north of the Main Central Thrust and south of the fossilifereous Tibetan - Teltys zone. Those crystalline metamorphic rocks representing the Kyanite - sillimanite bearing gneiss and schists, quartzite and marble extends continuously along the entire length of the country. At a few locations granites are found emplaced in the upper part of this unit. The northern or upper limit of this zone is generally found bounded by the normal faults. The zone is important only in the eastern Nepal from the aspect of hydropower development. The rocks inherent in this zone are observed generally hard, strong and competent. The existing Puwa Khola Hydro-electric Project and Tatopani Small Hydro-electric Project had been built in this zone. The location of the proposed Arun-3 Hydropower Project also falls in this geological environment.

Tibetan - Tellys Zone

The Tibetan - Tellys Zone is comprised of fossiliferous rocks such as shale, limestone, dolomite and sandstone ranging in age from Lower Paleozoic to Paleogene. It begins at the top of the Higher Himalaya Zone and continues to the north of Tibet. The regions such as Mustang, Manang and Dolpa as well as the Great Himalayan Peak like Mount Everest, Manaslu, Annapurna, and Dhaulagir fall in this geological zone. Due to presence of extremely rugged terrain condition having very steep slopes and deeply cut valleys, in this zone the development of the hydropower projects are difficult.

Thrust

The thrust are considered to be the regions in the ground through which not only movement of the rock strata has taken place but also the inherent stress was released during the geological history of the rock mass. The thrust are generally accompanied by the shear zones made of either broken or brecciated rock zone of high permeability and low strength, or clay zone of low permeability and remarkably low strength. As a result, any excavation through such regions will be susceptible to ground failure and release of the stress and ground displacement. In case of the underground excavation, failure of the rock adjacent to the thrust / shear zone can lead instability of the ground in the form of either gradual closure of the excavation or roof falls or squeezing whereas the surface excavation can create landslide and slope instability in such regions.

In consideration of prevalence of unfavorable rock characteristics around the thrust zone, surface geological investigation should establish their location, continuity and rock quality useful for the design of the conveyance system. Surface geological investigation should deduce the likely geological characteristics of the thrust for the underground conveyance locations. In many cases the thrust may not be clearly apparent on the ground surface right at the conveyance route, and in such situations the information should be obtained from the surrounding region. Three major thrusts having generally east-west trend are known in the country. Those thrusts observed from the south to the north direction are namely (i) the Main Frontal Thrust / Himalayan Frontal Thrust, (ii) the Main Boundary Thrust, and (iii) the Main Central Thrust. Their nature and characteristics are presented below for the purpose of design considerations.

Main Frontal Thrust: Generally the Main Frontal Thrust (MFT) is found buried under the sediments of Pliestocene to Recent age but in some location, its position is exhibited as overriding of the recent sediments by the Siwalik rocks. Based on the study in field and of the aerial photograph, the MFT are reported to be active at 17 locations in the country. It is apparent that the MFT will have virtually no influencing role in construction of the hydropower conveyance system in Nepal by reason of their position outside the hydropower potential zone. In consideration of this fact, the study of the MFT in reference to the design of the hydropower conveyance system will be less important.

Main Boundary Thrust: The Main Boundary Thrust is a fault zone which has brought the Lesser Himalayan rocks over the Siwalik rocks. It is either exposed on the ground surface or covered under the Quaternary sediments. It is considered to be active at 8 locations. Its annual slip rate is reported as 0.75 to 1.0 mm at the Bheri fault. Sub-surface information about the characteristics of the Main Boundary Thrust is virtually absent. None of the existing 14 hydropower projects built in the Lesser Himalaya Zone are located across the MBT but they lie to the north from the MBT. In practical sense, the MBT occupies the region outside of the hydropower potential zone however some of the future hydropower projects such as the Sun Koshi - Kamala diversion Project and Sharda - Babai hydropower Project will pass through the MBT. As to date the sub-surface geological information about the characteristics of the MBT is virtually absent. Surface geological information should be extensively assessed to predict accurately the sub-surface rock quality at the respective hydropower project. Supplementation of sub-surface geological investigation by conducting drilling and driving test adits will be necessary for the sub-surface conveyance structures through such zone.

Main Central Thrust: The Main Central Thrust (MCT) running almost in the east-west direction differentiates the overlying high grade metamorphic rocks from the underlying meta-sedimentary rocks of the Lesser Himalaya Zone. It occurs either exposed on the surface or buried under the Quaternary soils. It is reported active at 4 locations in the country. The reported Talphi fault with an average uplift rate of 0.2 to 2.5 mm/year is one of the active faults belonging to this category that occurs in the Western Nepal.

The existing 3 hydropower projects namely the (i) Tatopani Small Hydro-electric Project, (ii) Lower Khimti Hydro-electric Project and (iii) Puwa Khola Hydroelectric Project constructed in the Higher Himalaya Tectonic zone do not pass through the MCT hence information on their sub-surface characteristics are not available. In true sense, the MCT have been identified at many locations in the field and subsequently marked on the geological maps but the rock quality inherent at the MCT is never described in the form useful for the engineering design purpose of the hydropower conveyance system. Hence, the geotechnical investigations to be carried for the future hydropower projects around the MCT should be oriented to obtain first hand information and characteristics of the shear zone, rock quality around the thrust, presence of stress releasing features, occurrence of soft, squeezing and loosening ground.

Shear Zone: The shear zones are the common features produced as a result of thrusting and faulting activities in the ground of the Himalayas. In general, the shear zone indicates either the weak, soft and fractured rocks of high permeability and low strength nature or the clay zone of remarkably low permeability and low strength contributing major role in resulting instability of the ground. Information on nature / characteristics of the shear zones existent in the different geological units are scanty. They are generally overlooked or not recognized on the ground surface due to buried under the soil cover during the geotechnical investigation. Therefore in any cases, traverse along the gully and river courses will be helpful information on likely presence of the shear zone and its characteristics around the project area. Such information obtained either from the existing surface and underground conveyance system or from the geotechnical examination in the test adits or from the under-construction hydropower projects in similar geological condition may also be useful for design consideration.

Some of the available information is presented in the table below to provide opportunity to acquaint with the characteristics of the shear zones existent in the different tectonic regions of the country.

Shear Zone Observed in Different Tectonic Zone

S. No.	Project	Width of the shear zone observed in the tunnel / test adits	Tectonic Zone	Remarks
1.	Karnali - Chisapani Multipurpose Project	Presence of shear zones of 1 cm to 10 cm thickness in claystone, mudstone, siltstone and sandstone.	Sub-Himalayas / Siwalik zone	The shear zones were observed in the test adits dug for 672 m length.
2.	Sapta Gandaki Hydroelectric Power Development Project	Occurrence of two shear zones of 0.4 m and 5 m width in claystone, mudstone, siltstone and sandstone.	Sub-Himalayas / Siwalik zone	The shear zones were encountered in the test adit driven for 100 m length. The rocks were categorized as Class - C (weak to medium hard) with seismic velocity of 1.7 to 3.6 km/sec.
3.	Pancheswor Multipurpose Project	Association of shear zones of up to 2 m thickness in schist, quartzite, gneiss and granite.	Lesser Himalayan Zone	The shear zones were established based on observation in the test adits excavated for 614 m length.
4.	Melamchi Drinking Water Project	Inheritance of shear zones of up to 6 m thickness in schist and quartzite.	Higher Himalayan Zone	The shear zones were identified in the Likhu test adit driven for 96 m length.
5.	Kulekhani - II Hydropower Project	Existence of shear zones of up to 0.45 m width in schist and quartzite	Lesser Himalayan Zone	The shear zones were confirmed based on examination of tunnel penetrated for 5847 m length.

The shear zones are the unfavorable geological features for construction of the structural components of the hydropower project as it results creation of difficult condition, involvement of increased cost and longer time requirement in project construction and even failure of the project. In consideration of above fact, the shear zone should be properly identified and assessed for deriving information in terms of spatial position, characteristics and risk involved so that appropriate designing of the hydropower conveyance system could be achieved.

2.1.4.4 Safety Considerations

Safety consideration during design of water conveyance system, particularly of high pressure type, at the locations where it has to cross the mountainous torrential rivers / rivulets is very important. The mishap occurred at the over ground crossing of penstock pipe of Kulekhani-I hydroelectric project at Jurikhet rivulet during the disaster caused from the debris flows brought by incessant rain of 1993 warns the water conveyance designers to be very cautious. Hence, during the field visit the field team liked to visit the location where this disastrous event occurred in order to gather lessons. Unfortunately, the security guards did not allow the field team to visit the place despite the field team has letter from the concerned authority(s). Anyway, some description of the mishap from secondary source could be of great value for consideration during preparation of the guidelines. Hence, the following have been depicted from the sources available in the project office in the field.

The 1993 Flood Damage Assessment Report of October, 1993 Snowy Mountains Engineering Corporation Ltd. in association with CEMAT Consultants indicates following damages which have relevance with water conveyance system in the Hydropower Projects:

- Horizontal metallic penstock pipe of Kulekhani No. 1 with supporting structure;
- Intake structure, diversion dam, desanding basin at Mandu Khola headworks site for Kulekhani-2 were completely washed away requiring new design;
- Intake pondage for Kulekhani-2 and tailrace tunnel for Kulekhani-1 filled with gravel and sand-requiring cleaning.

The Photo Album with Introductory Note published by Water Induced Disaster Prevention Technical Centre (DPTC), Ministry of Water Resources in December 1993 to illustrate disaster of July, 1993, states the following:

"The major hydropower plant of Kulekhani-I and Kulekhani-II suffered severe damages. Debris flows caused by the incessant rain brought thousands of tons of boulders down the mountain stream and broke away the penstock pipe at Jurikhet Khola causing stoppage of Kulekhani-I system. The intake of Kulekhani-II at Mandhu Khola was also damaged by the debris flows. A boulder as big as 4000 tons of weight (about 20 m X 10 m X 8 m) flowed down the Mandu Khola to near the Mandu intake. This happened at a time when the people were having to bear power shedding owing to only 203 MW of power being produced against the demand of 250 MW. The damage to the Kulekhani hydropower system meant a short fall of 40% in the national power system".

2.1.4.5 Others

Social, Cultural and Environmental issues raised by the stakeholders should be fully addressed during the design consideration. Any negligence to response on such issues will invite unnecessary problem during the construction phase.

2.1.5 Environmental Aspect

2.1.5.1 Introduction

Environmental studies (Initial Environmental Examination or Environmental Impact Assessment) of development projects has been the legal requirement in Nepal since promulgation of Environmental Protection Acts, 1997 and its Rules in 1997. The Schedule 1 and 2 of Rule 3 of EPR 1997 details out proposal requiring IEE and EIA level of studies respectively. The hydropower development of size 1 MW to 5 MW would have to go through IEE process and hydropower projects having more than 5 MW installed capacity would have to go through EIA process. In addition any project which is located within the sensitive area such as National Parks, Conservation Area, Buffer Zone etc will have to go through EIA process. Water Conveyance Systems being the part of hydropower development both EPA and EPR are attracted.

2.1.5.2 Environmental Process Guideline

National Environmental Impact Assessment Guideline, 1993 is the first formal guideline on environmental study in Nepal. Environmental Protection Act, 1997 and Environmental Protection Rules, 1997 are the legal documents which have made the environmental protection as the legal requirement in implementation of the development projects. The Rules were amended in 1999.

In the IEE process the Terms of Reference for the Study will have to be endorsed by the concern ministry. In the case of hydropower project, the concern ministry is Ministry of Water Resources. IEE study will have to be carried out with the active participation of the stakeholders of the project which are project affected people and the local institutions. The peoples participation have been ensured not only by taking out 15 days public notice in the national daily newspaper but also making the deed of public appraisal (muchulka) of the notice in the project area and the collection of recommendations from the local bodies (VDCs/municipalities).

In the project requiring EIA level of study, the scope of work of the EIA study is determined with the active participation of the stakeholders which include the project affected people. The participation of the stakeholders in the scoping exercise is ensured by the public notice in the national daily newspaper and collection of issues and suggestions from the VDCs/municipalities of the project area. Based on the Scoping exercise, terms of reference for the EIA study is drawn and get it endorsed from Ministry of Environment, Science and Technology after the passing through review committee comprising of number of eminent environmentalists.

EIA study is to be carried out with the active participation of the stakeholders. The EIA study will have to cover four major environments namely: physical, biological, socio-economic and cultural. It suggests presenting the baseline information of the project area in these four environments. Identify the environmental parameters under these four environments, predict and evaluate the impacts due to the project implementation at the different phases namely: pre-construction, construction and post-construction. Each of the identified adverse impact will have to be mitigated or compensated whereas the possible and practical benefit enhancement measures will have to be proposed for the beneficial impacts. In order to ensure the implementation of the proposed mitigation measures, the environmental management plan is the basic requirement in the EIA study which includes the implementation mechanism of the proposed mitigation measures and environmental monitoring plan for ensuring the implementation of the mitigation measures and assessing the effect of mitigation measures. The public participating in the EIA study has been the major thrust. Public hearing of the EIA findings in the project area and recommendation from the local bodies (VDCs/municipalities) is the mandatory requirement in the EIA process.

2.1.5.3 Data Collection

Baseline data collection could be made through different methods. In general the baseline data are collected by the walkthrough survey, inventory survey, measurement, sampling, focused group discussion, household survey, local enquiry etc.

2.1.5.4 Identification of Environmental Impact

Environmental impacts can be categorized under four major environmental setting namely: physical, biological, socio-economic and cultural. The impact can be categorized as direct, indirect and cumulative. The impacts could be identified by different methods. The simplest methods would be Checklist Method, Matrix Method and Network Method.

2.1.5.5 Impact Prediction

The impact prediction will have to be made on the basis of available baseline data on the environmental condition. Such anticipated changes or impacts will have to be described in quantitative and qualitative terms. For predicting the environmental impacts at least following will have to be undertaken:

- Determine the initial or basin environmental condition
- Predict the future environmental condition in case the concern project is not implemented
- Estimate the future environmental condition in case the concerned project is implemented

In the process of impact prediction, magnitude, extent and duration of the impact will have to be assessed.

2.1.5.6 Mitigation Measures

While proposing the mitigation measures for the identified impact, one has to consider whether the alternative for the activity is available or not. If the alternative is not available what would be the corrective measures that would lessen the impact or is there any preventive measure that would lessen the adverse impact. The compensatory measures would be the last option that will be considered in the mitigation measures.

2.1.5.7 Other Guidelines and the Manuals to be followed for the Hydropower Projects in Nepal

The following guidelines and manuals relevant for environment need to be followed to fulfill the Environmental requirements in the development of Hydropower Projects in Nepal.

- National EIA Guidelines, 1993,
- EIA guideline for Forestry Sector, 1995,
- Guidelines for Environmental Monitoring and Environmental Auditing of Water,
- Energy Projects in order to integrate environmental aspects in the water resources Projects (WECS, 1995).
- Forest Produce Collection and Sales Distribution Guidelines (1998)
- Manual for Preparing Scoping Document for Environmental Impact Assessment (EIA) of Hydropower Projects
- Manual for Preparing Terms of Reference (ToR) for Environmental Impact Assessment (EIA) of Hydropower Projects, with notes on EIA Report Preparation
- Manual for Preparing Environmental Management Plan (EMP) for Hydropower Projects
- Manual for Reviewing Scoping Document, Terms of Reference (ToR) and Environmental Impact Assessment (EIA) reports for Hydropower Projects
- Manual for Preparing Initial Environmental Examination (IEE) Report for Hydropower Projects
- Manual for Public Involvement in the Environmental Impact Assessment (EIA) Process of Hydropower Projects
- Manual for Developing and Reviewing Water Quality Monitoring Plans and Results for Hydropower Projects
- Manual for Prediction, Rating, Ranking and Determination of Significant Impacts in Environmental Impact Assessment (EIA) of Hydropower Projects.

2.2 Hydraulic Models

2.2.1 Introduction

Hydraulic models have been used for more than a century for planning and design of hydraulic parts of hydropower plants, bridges and other civil engineering works in rivers, and for coastal and offshore structures as well.

The term “physical model” is rather new in connection with hydraulic modelling. It has come into frequent use in literature as a means to distinguish laboratory-based models from mathematical models. A more descriptive expression also in use is “laboratory model”.

In a physical model, flow and pressures are simulated by means of fluids in a small-scale version of the topography or structure to be studied (the “prototype”).

Osborne Reynolds designed one of the first known scale models in 1885 at Manchester University for study of tidal flow in Upper Mersey. In 1898 the River Hydraulics Laboratory in Dresden was established as the first of a long series of similar laboratories. The most active period in Europe and the USA occurred from 1920 to 1980, when most universities or institutes dealing with river or coastal hydraulics operated their own laboratories. Since 1980, high operational costs, strong competition and introduction of numerical models have gradually reduced the activities in many model laboratories. Development of locally based laboratory facilities in developing countries has become a new trend, as a result of the fact that most new hydropower projects will be constructed in these countries in the coming decades.

The use of hydraulic models has its background in two facts:

- Turbulent water flow is extremely complicated to analyze by theory. Empirical formulae exist for simple cases, e.g. for straight channels and pipes. However, natural watercourses as well as civil engineering structures usually include curved or irregular boundaries, such as constrictions, expansions, diversions, variable slopes, etc. Application of empirical formulae to such cases can only give approximate answers, usually not satisfactory for optimal design of civil engineering structures. Some examples are analysis of flood levels, erosion and deposition of sediments, and design of bank protection works.
- Most hydraulic structures in natural water-courses will need to function for a wide range of relevant parameters, including normal flow situations as well as floods, ice formation, sediment deposition and scour. It is impossible to wait for construction, or arrange site conditions that cover the full range of conditions and parameters needed to document the success of the structure.

By constructing a topographical model of a river reach or structure, it is possible to study the main flow parameters, such as flow pattern, slope, and velocities, in an analogue and visual way. Parameters may be directly measured in the model or documented by flow pictures. Local design and construction details may be changed and visually evaluated before the final design is documented in full.

A model will always represent some simplification of the full-scale case. Nevertheless, if certain empirical rules for interpretation of model tests are applied, it may give results with a high degree of accuracy and reliability.

These rules are related to three factors:

- the applied model law,
- the choice of model scales,
- the representation of model boundaries.

The scales and model boundaries have a great effect on the cost of the model, and the final design of a model will therefore usually represent a compromise between cost and accuracy.

2.2.2 Problems Suitable for Laboratory Modelling

Typical hydraulic problems suited for laboratory models include flow of turbulent water through open or closed conduits with fixed boundaries. For flow carrying sediments, ice, floating debris or air bubbles, special techniques have been developed.

In hydropower design, modeling has been in extensive use as a tool for investigation of problems related to the large construction works planned in projects on natural water-courses. It is necessary to optimize the design of such large structures both for cost-efficient production and to minimize environmental impacts. Traditional hydraulics based on empirical formulae, together with the designer's past experience, will usually be sufficient for arriving at a preliminary basic design. However, many structural details can only be optimized by the trial and error method, using a physical model.

Until sufficiently powerful computers became available, laboratory models were the only realistic tools for study of alternative designs and varying flow parameters. In consequence, the limits for operation of physical models were stretched, and modeling techniques were developed for a wide range of problems, sometimes wider than the original principles of hydraulic modeling were actually meant to handle.

In hydropower projects where maximum upstream flood levels should be determined, the design and capacity of the flood spillways soon became an important issue for hydraulic testing. The developers wanted to minimize the gate and spillway costs without risking compensation claims for flooding of upstream landowners.

Other important issues for hydropower models are:

- head loss reduction in feeder and tail-water channels
- design of intakes in order to minimize vortices
- control of surges and governor stability -
- measures to control drifting trash or ice
- stability and overtopping of embankment dams

Using a typical run-of-the-river and / or derivation type hydropower plant as an example, problems studied in models might include:

- the overall arrangement of dam, gates and intakes
- backwater and tail-water levels under normal and flood conditions
- detailed design of gates, head-race canal, intake and tail-water
- capacity of flood spillways and bottom sluices
- design of stilling basins downstream of the spillway
- precise calibration of dam gates and bottom sluices for operational use
- operation of gates during various discharges and production pattern
- operation of intake controls during various discharges and production pattern
- passing of ice or drifting debris
- fish ladder location and operation
- dynamic pressures on gates and other structures
- surges due to load variations
- stability of banks and riverbed against erosion
- safe dimensions for artificial bank and bed protection
- sediment accumulation and removal at reservoirs and intake ponds
- design and operation of sediment excluders

In hydropower schemes involving transfer of water through shafts and tunnels, models have been used to study:

- design of brook inlets
- analysis of transition areas between open surface and closed flow
- formation of air pockets
- surges and surge chambers

Mechanical equipment such as turbines, valves etc are usually studied in special laboratories equipped for model testing at high pressures.

2.2.3 Hydraulic Laboratory Models

2.2.3.1 Full Scale Models

The only way to obtain full similarity between a prototype and a model is to construct the model to full scale. This is in most cases quite unrealistic for studies of large civil engineering schemes. For small details, however, it may in some cases be feasible to test full-scale copies in a laboratory, because all parameters will be subject to free choice and under full control in laboratory conditions. In nature discharge, temperature and weather conditions will vary out of control, and the most interesting cases, e.g. extreme floods, are unlikely to appear at all during the period of study.

Few hydropower details are suitable for full-scale studies. Some examples are trash rack bars, flow aerators, governor guide vanes for turbines, valve details etc. Even these details can usually be studied successfully in a moderately reduced scale model.

2.2.3.2 Scaled Models

Most physical models apply some set of reduced scales. This means that some less important parameters must be neglected, while the more important parameters will be scaled according to “model laws” that realistically simulate the most relevant factors for the problem. Scales for different parameters have to match the physical laws. After choosing one scale, for instance the length scale, the other scales will be fixed automatically by the applied model law. In hydropower engineering, the flow will usually be turbulent and be driven by gravity as the single important force. The two most relevant parameters to represent in a model are then geometry and water flow. Friction is also important, but is usually a direct effect of the form and structure of the geometric boundary. Effects from viscosity or surface tension can usually be neglected. When this is the case, a set of scale ratios can be derived from Froudes model law, and the important effect of gravity on water flow is modeled accurately. Long experience has developed criteria for deciding when errors due to the neglected factors are small enough to be acceptable.

In very shallow or nearly non-turbulent water, surface tension or viscosity can not always be neglected, even if Froudes law applies to the rest of the model. In a model built according to Froudes law, such regions must be treated separately by special techniques. One much used trick to reduce the effect of surface tension is to spray the surface with detergent. In such a way it is possible to reduce surface tension effects without influencing the flow in deeper areas. A disadvantage is that unwanted foam sometimes may appear below areas of energy dissipation.

In some cases gravity is unimportant due to a near horizontal water surface or external pressures. Model scales can then be derived from Reynolds’ law for viscous conditions, or from Weber’s law for surface tension. The three model laws give very different sets of scales, and can therefore not be combined in one model. When planning to use a physical model it is, therefore, necessary to decide which of the three laws is most relevant for the problem to be studied.

2.2.3.3 Distorted and Undistorted Models

If the same geometric scale is used in all three dimensions, a normal undistorted model will result. This is the most common situation for physical models, and the simplest models to operate and analyze.

However, many rivers are wide and shallow. An undistorted model will then either result in very shallow water flow, if the model scale is small, or a model with very large horizontal dimensions, if a large enough scale is chosen to give enough water depth. As a compromise, it is sometimes possible to use different scales for horizontal and vertical lengths. This gives a distorted model. It requires special handling of some parameters, mainly the bed friction, but used by experienced personnel, it can improve the results of modeling of shallow water bodies and rivers.

Models of tunnel systems with shafts and branches will often require distortion because of disproportion between tunnel diameters and tunnel lengths.

2.2.4 Model Laws for Scaled Models

2.2.4.1 Common Principles

Model relations are derived from dimensionless combinations of relevant parameters. Any dimensionless equation may in principle be used to create a model relation, if we insert their scaled-down values instead of the real parameters.

All model studies using reduced topographical scale will have to neglect some less important parameters in order to simulate the more important parameters. In hydraulics of incompressible fluids, three dimensionless numbers are particularly important, because they relate velocity and length to the influence of gravity, viscosity, and surface tension respectively. Using *V* for velocity, *L* for length, g for acceleration of gravity, ρ for fluid density, υ for kinematic viscosity, and σ for surface tension, these numbers are:

Froude number: $Fr = V / \sqrt{g \cdot L}$ ------------------------------- (2.4)

Reynold number: $\mathrm{Re} = V \cdot L / v$ ------------------------------- (2.5)

Weber's number: $We = \rho \cdot V^2 \cdot L / \sigma$ ------------------------------- (2.6)

By far the most common model law used in hydraulic modeling is named Froude model law. It relates gravity and inertia forces, neglecting viscous and surface tension forces. Model laws based on Reynolds' or Weber's numbers may be applicable in some special cases. But this is not discussed further in this guidelines because of the limited relevance of such modeling in hydropower engineering.

2.2.4.2 Froude Model Law

Froude model law is obtained by equating the model and prototype versions of Froude number. Using index "*p*" for prototype, "*m*" for model, and "*r*" for scale ratio, the length scale ratio is given as

$L_r = L_m / L_p$, the following equations result:

$$V_p / \sqrt{g_p \cdot L_p} = V_m / \sqrt{g_m \cdot L_m} \qquad \text{--------------------------------} \qquad (2.7)$$

$$V_r / \sqrt{g_r \cdot L_r} = 1 \qquad \text{--------------------------------} \qquad (2.8)$$

The model is subject to normal gravity, hence $g_r = 1$, and the model law reduces to $V_r = \sqrt{L_r}$

In words: the velocity scale becomes equal to the square root of the length scale. Other scales are found by combining L_r and V_r, (*T* = time, *Q* = discharge, A = cross-section area):

Parameter	Formula	Scale ratios in terms of L_r	Example: L_r = 1/50	Example: L_r = 1/1 00
Length	L	L_r	L_r=1/50	L_r= 1/100
Velocity	V = L/T	$V_r= L_r^{1/2}$	V_r=1/7.07	V_r= 1/10
Time	T = L/V	$T_r=L_r/L_r^{1/2}=L_r^{1/2}$	T_r=1/7.07	T_r= 1/10
Discharge	$Q=AV=L^2V$	$Q_r=L_r^2V_r=L_r^{5/2}$	Q_r=1/17678	Q_r=1/100000

Two important points can be noted:

1) Velocity and time will both be reduced less than the length, such that the model flow visually will appear swifter than in the field.

2) The scaled discharges are small enough to allow for simulation of comparatively large floods using commercially available pumps. (A 1000 m^3/s flood requires only 10 l/s in a 1:100 scale model and 56.5 l/s in a 1:50 scale model).

Froude law is also the basis for distorted models, using different horizontal and vertical length scales. Distorted models can only be used for rather simple topographic situations and flow situations. The velocity scale will relate to the vertical length scale H_r, i.e. $V_r = H_r^{1/2}$, since gravity acts vertically. The discharge and time scales become more complicated: $Q_r= L_r\ H_r\ V_r=L_rH_r3^{1/2}$ and $T_r=L_r/H_r^{1/2}$ for the horizontal motion.

Oscillations in tunnels are driven by gravity, and Froude model law will usually apply. However, because of disproportion between tunnel diameters and tunnel lengths, models of tunnel systems with shafts and branches will usually require distortion. In a period before numerical models came into use, some complicated tunnel systems were therefore modelled according to a special set of equations called Durand's similarity rules. These rules employ Froude law, but allow for distortion of both diameters and tunnel geometry. Textbook references are not known, but examples may be found in old reports.

2.2.5 Problems Requiring Special Modelling Techniques

2.2.5.1 By Passing of Logs and Trash

Log floating was once an important means of transporting timber, and old rights of passage had to be taken care of when constructing dams or diverting water. Design and testing of log by-pass often required tedious model studies in rather large models. Length scale 1/50 was quite common for run-of-the-river plants. The logs could be scaled by the geometric length scale and made of wood, since the density scale was 1:1 as for the fluid.

In recent models, the same scaling rules have been used for study of trash passage through spillways, gates and bridge openings. Trash accumulation has come into focus as a potential cause of reduced spillway capacity due to clogging during major floods, resulting in serious dam safety problems.

2.2.5.2 Ice Problems

Ice has caused much trouble for run-of-the-river plants in High Mountain areas and other cold places. The problems are of two types: those caused by accumulation or passing of drifting ice in a passive state, and those connected to ice formation under freezing conditions.

The drifting ice can be modelled according to similar principles as logs, but model tests using real ice are difficult to operate. Instead, many model studies have successfully used artificial materials of the same density as ice, usually plates of wax or plastic, cut or chopped to suitable dimensions to simulate floating ice of various types from blocks to small ice fragments or even small lumps of frazil.

Problems with freezing ice can not be studied in models at normal indoor temperatures, but some flume studies have been arranged in cold rooms in order to analyze some of the complex principles which apply to ice formation and movement in rivers under sub-zero ambient conditions.

2.2.5.3 Oscillations

Transient problems such as oscillations in intake ponds, tunnels and surge shafts can be studied in physical models, but special rules for distortion are usually necessary when long tunnels with normal diameters are involved. Very spectacular physical models were once constructed for simulation of surges in tunnels with many communicating shafts. This type of problem was among the first model problems to be analyzed by one-dimensional numerical programs, and laboratory models of this kind are not in use any more.

2.2.5.4 Sand in Intake Ponds and Sand Traps

Heavy sediment load is less during dry season but they are high in monsoon, and extreme floods. The riverbed will therefore usually be rather stable under normal flows. Natural riverbeds may undergo severe changes during extreme floods, however, and local bed changes may occur near structures placed in the river.

Much effort has been made to find reliable methods for the study of flushing and diversion of sediments in models. Sediment handling may be decisive for operation and economic lifetime of a power plant. Sediments will accumulate in intake reservoirs unless removed by hydraulic or mechanical methods. Suspended sediments passing through the power plant intake need to be settled out in sand traps and diverted before reaching the turbines.

The model scale for particle size D_r depends on both the length scale L_r and the scale of submerged density of the particle $(\rho_s - \rho_w)_r$, where ρ_s and ρ_w are the densities of particle and water respectively. For particles moving along the bed the expression for diameter scale becomes.

$$D_r = L_r / (\rho_s - \rho_w)_r \qquad \text{--} \qquad (2.9)$$

In steep rivers with mainly coarse material, both moving material and river bed particles may be simulated by natural sandy material of the same density as in nature. In this, case the density scale is 1:1 and the diameter scale becomes equal to the length scale.

This is not possible with finer material, however, because the scaled-down particles will be so small that the similarity is lost. The main reasons for this is

- particles of silt size and finer are bound together by cohesive forces
- particles smaller than the boundary layer thickness of the flow are not simulated correctly.

In practice the smallest model particle size is usually around 0.2 mm. This means for example that natural material can only be used to model gravel sediments larger than 10 mm in a model with length scale 1:50. A method to overcome this problem is to use particles of lighter material than sand. Frequently used materials are various plastics, crushed coal etc.

For a plastic material of density 1050 kg/m the submerged density ratio $(\rho_s - \rho_w)_r$ in relation to sand of density 2650 kg/m^3 will be

(1050-1000):(2650-1000) = 1:35.

In a model with length scale of 1:50 as example, then

$$D_r = L_r / (\rho_s - \rho_w)_r = (1:50) / (1:35) = 1:1.4$$

allowing for modelling of natural sand, using larger plastic particles. In that way both cohesion and boundary layer problems mix be avoided, but other problems arise. The exaggerated size of the artificial particles influences the bed roughness, and the usually rounded form of plastic particles creates a very unstable bed in the model, affecting the stability of slopes etc.

2.2.5.5 Air Accumulation in Tunnels

Intrusion of air into hydropower tunnels is known to cause several problems. e.g. head losses, explosions due to escaping compressed air, and nitrogen absorption. Because air is compressible, air is difficult to simulate in scaled models. Some local problems have been studied with good results, such as air accumulation in tunnels just downstream of steep shafts with air intrusion. These studies have guided the design of many brook intakes in order to reduce intrusion of air or violent escape of accumulated air.

Model studies have also been used to document how elongated, unstable air pockets may form under the tunnel ceiling, causing extra head loss and sometimes unstable flow conditions.

2.2.6 Model Calibration

Calibration is an important part of a model study. Neither physical nor numerical models can usually give correct answers without initial adjustments in order to bring the model result into correspondence with known values from the prototype.

In a physical model, built for one particular case study, calibration will usually involve known combinations of water level and discharge, and adjustments will include small modifications of boundary conditions such as surface roughness or downstream water levels. The changes can be introduced manually in the model.

A numerical model study will usually be based on a commercially available computer program which has to be adjusted to apply to the problem in question by introduction of relevant boundary conditions. Calibration of the applied model version can be done by using the same type of available site data as for calibration of a physical model.

A general numerical program, on the other hand, will not have a particular prototype, but be made to suit a variety of related problems. During development, it is necessary to test or “calibrate’ the program against real cases. In combination with data from real field cases, a laboratory model may conveniently be used for this calibration, since it offers a possibility for introduction of a great variety of parameter sets. Such use of laboratory models, either built for the sole purpose of developing the program, or being available later for a project test, could be an important task for hydraulic laboratories.

2.2.7 Principles of Numeric Modelling

2.2.7.1 Numerical Methods

Partial differential equations, like Saint-Venant’s equations, are usually impossible to solve analytically, so numerical methods are often used. Two main approaches are the finite difference method and the finite element method. The finite difference method is discussed here as it is commonly used in one-dimensional hydraulic computer programs. In this method the idea is to discretise the differential equations to make algebraic equations. For example, if we have a function *f* varying in time and space (t and x), we may write

$$\frac{\partial f(x,t)}{\partial x} \approx \frac{1}{h}\left[f(x+h,t) - f(x,t)\right] \qquad \text{---} \qquad (2.10)$$

and

$$\frac{\partial f(x,t)}{\partial t} \approx \frac{1}{k}\left[f(x,t+k) - f(x,t)\right] \qquad \text{---} \qquad (2.11)$$

Where, h is the distance step size and k is the time step size.

Numerical methods are prone to suffer from instability in the calculations. Therefore, numerical schemes are designed to obtain stability and convergence as often as possible. This means that small perturbations in the initial condition, or small errors at any time, remain small at later times and that the approximate solution converges to the exact solution as the distance and time steps approach zero.

For many numerical approximations, certain conditions need to be fulfilled to assure stability and convergence. One such condition can be that the time step must be smaller than a certain value for a given distance step (i.e. a decrease of the distance step may demand a corresponding decrease in time step). If such a condition is not fulfilled, a simulation may “crash”, i.e. stop before it is executed, or the simulation may finish, but be inaccurate and produce results which include unrealistic oscillations.

On the other hand, numerical simulations may sometimes artificially yield a smoother distribution than is real. For example, large gradients in concentrations of a chemical may be underestimated due to so-called numerical diffusion, i.e. a flattening of such gradients due to the numerical interpolation which is a necessary feature of the model. When one pushes the software to its limits, one should keep in mind that some rather radical means are sometimes applied to avoid “crashing” and numerical oscillations, and this can make the results unreliable.

There are different numerical schemes for the finite difference method, and the schemes can be explicit or implicit, in explicit schemes, the function values and derivatives in each point in the new time step are found directly from the values in the preceding time step. Implicit schemes, however, use the functions known values in the preceding time step, but also the unknown values at neighbouring points in the new time step. Since the functions values at a given point are dependent on other unknown values, a set of equations is formed. This set of equations can be solved by matrix methods if the boundary conditions are known. The advantage of the more complicated implicit methods is that they are always stable, which allows for larger time and distance steps and hence less computation time.

2.2.7.2 1D-, 2D- and 3D-Models

1D-models simplify the reality by using the fact that the longitudinal velocity of river flow dominates the flow perpendicular to the main flow direction. The water velocity at any point in a given cross-section is then set to the discharge divided by the cross-sectional area. ID-models are the only ones which have been extensively used in practical applications of river flow simulations so far. Some of the first programs, like HEC-2 (now replaced by HEC-RAS), were for steady flow, i.e. without variation over time. Software like HEC-RAS, DAMBRK and MIKE 11 also allow for unsteady flow and even dam-break wave simulations.

2D- and 3D-models have been developed and used, and obviously produce more accurate results provided that sufficient input data is available. Especially when a river inundates floodplains, the flow cannot accurately be treated as one-dimensional. This is because the friction usually is much higher and the depth smaller on the floodplains, causing the water there to flow slowly and sometimes be inactively stored. 2D-models, and especially 3D- models, can simulate the turbulence in a satisfactory way using for example the advanced k-E-model, which is the most advanced turbulence model in current use by the engineering profession.

Other processes, like sediment processes or biological and chemical processes related to water quality can be modelled in 1D-models. However, more reliable results can .be expected to be found using 2D- or 3D-models. The problems in using 2D- and 3D-models are obtaining sufficient geometric data, the computational power required and the complexity of the models. However, the use of 2D- and 3D-models is likely to increase in the future. Examples of programs are the finite element Telemac software with 2D- and 3D- versions, finite volume models such as MIKE 21 from the Danish Hydraulic Institute and the 3D software SSIIM, developed at NTNU in Trondheim, Norway.

2.2.7.3 1D River Modelling

Since most engineering problems are still being solved with the use of the simpler 1D-models, this sub-section will describe only this type of river modelling. The procedure when using ID-models is usually data gathering, model set-up, calibration and simulation. The modelling task may, for example, consist of:

- the production of a map of inundated areas for a flood with a given return period,
- the downstream effect of rapid maneuvering of dam gates,
- assessment of flood mitigation measures,
- the simulation of a dam break wave,
- an evaluation of the impact that some structure causes on the river flow,
- water level/velocity simulations,
- ecological studies.

The necessary inputs to a typical 1D-model are cross-sections, boundary conditions and initial conditions. Cross-section widths are calculated for different elevations so that the cross-sections are made symmetric. The cross sections should be extracted so that they are representative for the river. The location of each section is such that the model assumption of linear transition in shape between sections most accurately represents the true river geometry.

Boundary conditions need to be applied in the upstream and downstream ends, and can be a constant value or a time series of discharge or water levels. Typically, one may assume constant water level where the river reaches the ocean or a lake and the upstream boundary condition is a time series of discharge, for example a flood being released from a dam. Also, information about structures like dams, weirs, culverts or bridges must be specified. An initial condition can either be specified as water levels and discharges at specific points in the river or the program may obtain a stationary initial condition.

Calibrations of the model against known events or data should be made whenever possible. Sometimes, one can use rating curves from water level gauges in a river, or flood level marks from a historical flood. Flood marks at different locations along a river from a historical flood can be used to assure that the water line can be reproduced by the model for a given discharge. This calibration is especially useful if estimates exist of the actual flood discharge. When calibrating, the main tuning parameter is often the Manning coefficient assumed at different reaches of the river. Also, if the river bed profiles are uncertain in some places, one may adjust them within the uncertainty in the calibration process. One may also in the calibration process wish to change how some structure is treated in the model.

One should avoid blindly believing the simulation results and keep in mind the uncertainties embedded in them. One source of error is the estimate of the Manning number, especially if the flow on flood plains or in densely vegetated terrain is simulated. In a 1D-model, it is important that the cross-sections are representative for the river. With a finite number of profiles, this will always be a source of error. When the flow is strongly unsteady, as in a dam break wave, one should be aware of especially large uncertainties in the results. In these cases, numerical oscillations in discharge are not unusual. The uncertainty is also large in steep rivers where large Froude numbers occur, especially when the flow gets supercritical.

2.2.8 The Future of Hydraulic Models

When numerical programs recently became available, offering a fast way to perform some types of calculations in time and space, some problem types were soon removed from the laboratory modelling agenda, because they were found easier to study by numerical models. Linear problems such as surges in tunnel systems and backwater levels upstream of dams were among the first to be applied to numerical models (before 1970).

At the ASCE conference “Modelling ‘75” in San Francisco, a wave of optimism about the nearly unlimited possibilities of numerical models was expressed and some participants advocated the thought that numerical models could entirely substitute the laboratory models in the near future. Others were more realistic, talking of combining the two techniques.

Much effort has since been made in order to apply three-dimensional numerical models to study of details of hydraulic river structures. Simple structures with well-defined boundaries like overflow sections, expansions and contractions and diverging or merging channels can now be analyzed for effects of friction and shape. Curvilinear flow where diffraction and secondary flow structures are dominant is still difficult to represent numerically.

A gradual reduction of the market for traditional model laboratories has occurred, both because the numerical methods have become useful, but also as a result of declining hydropower development in many parts of the world.

Combinations of numerical and physical modeling will be the standard option for many future projects. It is expected that numerical methods in the future will be used for the general analysis of a project, leaving details not applicable to numerical computations for physical modeling. The result is that numerical models will be used first in the planning and preliminary design process, while physical models will be used later to test and document the success of these designs, and make design modifications.

2.2.9 Benefits and Costs of Physical Modeling

2.2.9.1 Benefits

The following are the potential benefits of physical modeling:

- increased system efficiency;
- potential saving due to improvement in system design, reduced construction and / or operating costs, reduction of materials, lower maintenance requirements;
- improved system safety;
- confidence in the design;
- prolonging system life;
- reduce environmental hazards;
- advancement in science and technology.

2.2.9.2 Costs

Except for simple investigations of isolated features, the cost of model study is not small. The cost of a hydraulic model study depends on the size and complexity of the structure and the number of problems to be investigated. The cost also increases, if complex shapes are involved, if sediment transport or erosion is modeled, if sophisticated, detailed measurements are required or if multiple aspects of flow around the structure are investigated. The last case may actually require more than one model at different scales for accurate investigations.

2.2.10 When a Modelling is Required

In case the risks associated with the design justify the model study cost, a model study should be carried out. The designers, first, should seek the answers on the following questions before deciding whether model study is required:

- What is the possibility that the structural component will not perform adequately?
- What costs are associated with inadequate performance of the structure?
- What potential cost savings can be achieved as a result of a model study?

World-wise experience of hydropower development is more than a century long. The experience on canal type of water conveyance system is much longer (some tens of centuries). Thus, designs adopted for different components of water conveyance system are generally well tested conventional type. Many empirical formulas suited to compute the value for any specific application are available. But, sometimes, due to unique site conditions their use could be limited. The possibility of poor design is increased when the engineers cannot use a standard design or previous experience. A hydraulic model study can be performed to verify that the proposed design functions properly. The model may also be a tool to improve structure performance or to reduce anticipated construction costs.

CHAPTER – III

DESIGN GUIDELINES

3.0 DESIGN GUIDELINES

3.1 Hydraulic Design

3.1.1 Structural Elements Considered for the Guidelines

The structural components required for conveying the water for power generation depend on site conditions, since the layout is to be arranged considering the available flow, head, topography of river and vicinity including the geological conditions of the project area and specific sites where the needed structural components will be located. Taking into account that the preparation of design guidelines for headworks is already underway under a separate contract, with a view to avoid duplication, the structural components considered for the present study are as follows:

a. For high dam river type scheme, penstock pipe will be of the major concern,

b. For run-of-river projects with head concentration by derivation alone or both by dam as well as the derivation, the conveyance system will comprise of:
   - channel connecting the intake and desanding basin (intake or approach channel);
   - desanding basin(s) and flushing structures;
   - power conduit (open or closed canal, pressured or non-pressured tunnel, flumes, aqueduct, siphon, cross-drainage works as appropriate);
   - forebay or surge tank / surge-shafts;
   - penstock pipes (surface, buried or embedded in tunnel); and
   - tailrace (canal or tunnel)

c. For derivation type projects with storage reservoir, the conveyance system will comprise of:
   - pressured tunnel or pipes;
   - surge shafts;
   - penstock pipes (surface, buried or embedded in the tunnel)
   - tailrace (canal or tunnel)

3.1.1.1 Canal

General

The canals for hydropower projects are constructed mainly for the purpose to alter the gradient of the river to benefit hydropower production. Two essential parts of the canal in a hydropower system are (i) headrace or power canal and (ii) tailrace. If a headrace / power canal serves to convey water from the head-works to the forebay, the tailrace conveys water released from the powerhouse to the river. The power canal designed for a run-of-river type hydropower project has generally two sections: one between head-works and desanding basin also called an approach canal and the other between desanding basin and forebay. Hydropower could also be produced utilizing the head available at canal drops constructed for conveying water for other purposes, but they are done generally in water transfer projects involving larger discharges where even a small head available at the unavoidable structural or natural drop(s) can produce appreciable amount of hydropower. In Nepal only two examples of such plants in a smaller scale do exist: - one at Gandak Western Irrigation Canal designed for 15 Mw capacity (presently working only at nearly half of its installed capacity due to silt problem) and the other with 3.2 Mw capacity installed at the main canal of Sunsari – Morang irrigation project. Three such plants each of 100 MW capacities on irrigation canal(s) have been proposed by

1981 Feasibility Study of Koshi High Dam Project conducted by the Government of India (GoI). Since the water conveyance system designed for irrigation or other purposes lie outside the scope of present study, the elements of canal design described for the guidelines are purely for approach canal, power canals and tailraces. The prospects of hydropower canals are high in the hills and middle mountain physiographic regions of the country, but could be limited in size due to steepness and fragile geological conditions of the terrains along the river banks. For the larger capacity canals, the options of covered canal with rectangular section instead of a more efficient trapezoidal section (which approximates the shape of the semicircle, the most efficient hydraulic section with the greatest hydraulic radius) may become the ultimate choice.

The optimum channel design will be governed by the following five key principles:

- the velocity of the water must be high enough to ensure that suspended solid (sediments) do not settle on the bed of the channel and that plant growth is discouraged
- the water velocity must be low enough to ensure that the channel walls are not eroded by the flow
- the channel must be durable enough to resist destruction by storm run off, rock falls and land slips
- the channel must be cost effective – not larger than necessary
- In some special section of the canal where water table is high, a provision of flap valve may be considered to minimize the slab thickness.

Approach Canal

Approach canal starts just after the intake and ends at the desanding basin. Its dimension and shape depend on the discharge to be conveyed, prevailing topography and geology of the alignment. This portion of the canal is usually equipped with a gravel trap and an overflow spillway. During normal operation, the size of the gravel and discharge entering to the approach canal is controlled by the coarse trash-rack and intake gate/s hence it conveys design discharge plus flushing discharge to the desanding basin. But during the high flood period in case of failing to operate the intake gates, excess discharge will enter into the approach canal. Therefore, the capacity of this canal and its freeboard are designed to cater the discharge entering from the intake during the considered return period flood in river (most intakes / headwork components are designed for 1 in 100 years of return period flood). Approach side walls are designed with sufficient freeboard to convey flood discharge safely up to the side canal spillway. This spillway is designed to exclude excess water than the design discharge plus flushing discharge.

The gravel trap is constructed close to the intake in order to prevent gravel from getting into the approach channel. Main function of the gravel trap is to collect the bed load, smaller than the trash-rack opening size, entering through it to the approach canal. Gravel trap's location is governed by the site conditions, availability of flushing head and gravel carrying capacity of the approach canal. Its dimension depends on the flow velocity, gravel size and specific density of the gravel and it should be sufficient to settle and flush gravels passing from the coarse trash-rack. Gravel trap is generally designed to collect maximum of 12 hours gravel deposit. A flushing arrangement associated within the gravel trap is operated to flush out the collected gravels to the river. Flushing frequency is less during the low flow periods whereas continuous flushing is recommended during the monsoon. A gravel trap may be equipped with overflow spillway. More details about the gravel trap and its design principles are covered in the Design Guidelines of Headworks for Hydropower Projects. For a typical section of gravel trap refer Fig. 3.1.

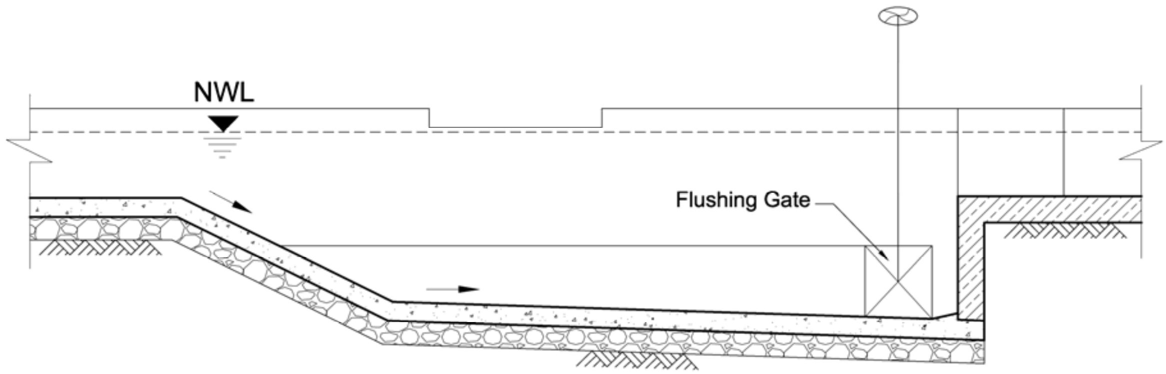


Fig. 3.1: Typical Gravel Trap

For presentation in a logical sequence of the project components, the headrace canal has been dealt with in Section-3.1.1.3 after desanding basin. Similarly, the tailrace has been dealt after penstock. The details of canal design are given in Section-3.1.1.3.

3.1.1.2 Desanding Basin

General

This guidelines deals desanding basin only because in a logical sequence it appears as a part of the system of a conveyance system. More details of the desanding basin is dealt in the design guidelines of headworks for hydropower projects which are presently under preparation by DoED under a separate consultancy contract. Where run-off-the-river plants are located on rivers which transport substantial suspended sediment loads, it is a general requirement that a desanding basin be provided to trap and exclude sediment particles in excess of a selected size so as to minimize damage to the turbine and its accessories.

The performance of desanding basin is guided by its ability to trap suspended sediments and its ability to remove the trapped particles from the basin, i.e. qualities of the adopted sediment flushing system. The main function of the desanding basin is to:

- maintain the hydraulic transport capacity of the waterways
- reduce the sediment load to the turbines,
- obtain the required power generation regularly.

Typical section, plan and elevation of a desanding basin are shown in Fig. 3.2 below.

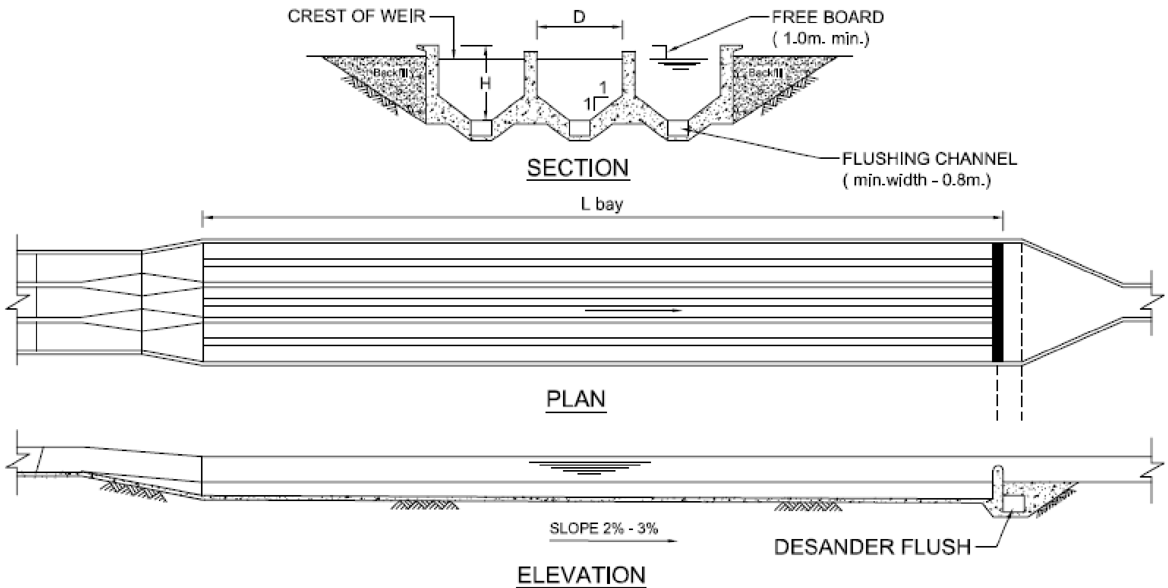


Figure 3.2: Typical Section, Plan and Elevation of the Desanding Basin

Design Consideration

Data Requirement: It is desirable that a regular program of suspended sediment sampling be initiated near the intake site from an early stage during site investigation to ensure that sufficient data is available for design. The sampling program should extend through the entire rainy season and should comprise at least two readings daily. On glacier fed rivers where diurnal flow variations may exist , the schedule of sampling should be adjusted to take this phenomenon into account and the scheduled sampling times be adjusted to coincide with the hour of peak daily flow with the other sampling taken about twelve hours later. A five year long sediment collecting program would be ideal and less than one monsoon season of data is considered unsatisfactory. Data normally collected in sediment sampling program would include:

- Mean daily concentration of suspended sediment
- Water temperature
- Flow in the river

The following additional information would be derived from collected samples:

- Particle size gradation curve
- Specific gravity of particles
- A sediment rating curve

Choice of Design Criteria: The rate of wear of turbine and accessories due to sediment abrasion is related to the following factors:

- Concentration of suspended particles
- Hardness of particles
- Size of particles
- Shape of particles
- Resistance of turbine parts
- Turbine head

These inter-relationships are complex and as yet there is no fully reliable method available for selecting the sediment size to be used for desanding basin design. Opinions of experts vary widely. Bouvard, citing European experience, recommends that no desanding basin are required for plants having less than 100m head, while Varshney recommends that all particles 0.2 mm and greater be excluded without regards for plant head (based on Indian Practices). In general for a medium head hydropower plants the removal of particles larger than 0.2 to 0.5 mm is usually specified. According to Sokolov, sharp edged quartzite sediment with a particle size as small as 0.25 mm may seriously damage turbines. For the high head plants particle size of 0.1 to 0.2 mm and even smaller might be objectionable. The wear of mechanical equipments installed at power plants with very high head may be prevented only by removing particles of size as small as 0.01 mm to 0.05mm. The smaller the particle size to be removed, the larger the desanding basin should be and vice versa, therefore, a design choice will have to be made based on largely on engineering judgment. In this regard a conservative choice would be recommended for plants on rivers having large annual sediment loads, while a less conservative choice could be adopted for plants on rivers with lower sediment loads. It is also advisable to make an economical analysis of the size of the desanding basin with respect to the turbine replacement / maintenance for the optimum size of the desanding basin for the given head, discharge and sediment concentration.

Operating Consideration: In general there are two types of desanding basin:

- **Continuous flushing desanding basin:** Continuous flushing desanding basin, uses surplus water for flushing, about 10%-15% of the plant discharge. This type of desanding basin does not interfere power production during the flushing process. During the low flow season when sediment inflows are minimal, flushing can be done intermittently, as required, so that most of the available water can be used for power production. However, the design and operation of continuous flushing desanding basin is more complex than for the discontinuous type and much more care is required in their operation. The main problem is clogging of the sediment extracting system, manifold openings (Dufour type), vortex extractors or hopper openings. In order to improve reliability and enhance operating flexibility, continuous flushing desanding basins are usually constructed with a minimum of two basins. Continuous flushing desanding basins are used in Sunkoshi, Adhi Khola, Jhimurk, Modi Khola, Puwa Khola, Chilime hydropower plants.
- **Discontinuous flushing desanding basin:** Discontinuous flushing desanding basins are of much simpler design and are much less susceptible to blockage or clogging. The main operating inconvenience is that plant output must be cut back, for multi basin design, or shut down entirely for single basin design (e.g., Marsangdi and Trishuli). In muddy water this is not easy to determine. A variety of instruments have been suggested, however, sounding from a bridge across the basin would seem to be more practical on small basins. Typically, discontinuous desanding basin release much larger flushing flows than continuous flushing type and do so suddenly. The desanding basin if functioning according to design will trap most of particles down to the target size and some proportion of smaller sizes, thus minimizing turbine abrasion. However, very fine particles of less dense minerals, such as mica will not be trapped. Flushing operation is conducted only when the desanding basin is full of silt.

A new type of desanding basin flushing system called “Serpentine” system –developed by Hakoon Stole at the Norwegian Institute of Science and Technology in Trondheim is being tried out at Andhi Khola, Jhimruk Khola and Khimti Khola hydropower plants in Nepal.

The main function of the desanding basin is to reduce the turbulence level in the water flow to allow suspended sediment particles to settle out from the water body and deposit on the bottom of the basin. The basic observation is that sediment particles, excepting very fine colloidal particles, will settle out in still water at a rate depending on the fall velocity of the particle. It will never be possible to trap all suspended sediment particles in the basin as the fall velocity of suspended silt and clay are too small compared with the turbulence level in the desanding basin. In essence then, a desanding basin should provide a sufficient retention time for a particle, to precipitate from the surface to the bottom of the basin.

The hydraulic design of a desanding basin arrangement shall secure:

- an even flow distribution between parallel basins,
- an even flow distribution internally inside each basin,
- efficient removal of deposits during flushing of the basin.

Basically there are two main approaches available to dimension the desanding basin with respect to the resulting trap efficiency of the basin:

Particle Approach: The particle approach to trap efficiency computation is assessing the probability of one particle being trapped or passed through the desanding basin. The particle approach is based on the simple relation. If there is no turbulence inside the basin, the ratio between the particles fall velocity, 'w', and the horizontal transit velocity in the basin, 'v', must be same as the ratio between the fall distance-'D' (depth) and the horizontal travel distance 'L' (length) in an ideal basin, i.e. a basin without any turbulence, all particles with a fall velocity larger or equal to 'w' will be trapped.

$$w = \frac{D * v}{L} \qquad \text{--} \qquad (3.1)$$

Where,

w	=	Fall velocity
D	=	Depth of the desanding basin
L	=	Length of the desanding basin
v	=	Horizontal transit velocity

Trap efficiency for this method is determined using Camp's diagram given in the Annex-1.

Concentration Approach: The concentration approach is the difference in average sediment concentration in the flow entering the basin and the flow leaving the basin. Trap efficiency of the basin is determined by the Vetter's method using following formula:

$$\eta = 1 - e^{-\left(\frac{w.A_s}{Q}\right)} \qquad \text{--} \qquad (3.2)$$

Where,

η	= Trap efficiency
A_s	= Net surface area
Q	= Discharge
w	= Fall velocity

For all simplified trap efficiency computation it is important to make reasonable assessment of effective surface area for settling. The net surface area is the area of the basin where the flow distribution is close to uniform. During the hydraulic design and dimensioning of the desanding basin, general practice is to ignore the effects of concentration on the rate of fall velocity of individual particles but the main parameter with respect to wear of turbine at high head run-of-river plant is the total amount of sediment load passing through the turbine and not the size of the individual particles in the sediment load. Standard design criteria for desanding basins are however linked to the basins' ability to trap particles of a given size. It is relatively easy to prove that a design is satisfying the trap efficiency criterion, which is a more or less direct function of the size of the desanding basins. This design approach is popular due to lack of sufficient data required for the concentration approach design of the desanding basin. This approach is not strictly true but for concentrations below 2000 ppm the effect is minimal and may be ignored. However, this factor should be taken into account for desanding basin operating on river carrying suspended loads of more than 2000 ppm for a significant portion of the time, in accordance with concentration approach.

Removal of Deposited Sediment

It is necessary to provide some dead storage in desanding basin where sediment may accumulate between the flushing processes. The size of the dead storage is dependent on the sediment load as well as the adopted flushing method. Two basic methods (continuous flushing and discontinuous flushing) mentioned above are used for the removal of the deposited sediment from the basin.

The sediment concentration in Nepalese rivers particularly during rainy season is very high, due to which even surplusly designed desanding basins have not been able to settle down fully the designed size particles. For the projects with larger design flow and lower head the costs of desanding basins are quite large and, therefore, it is cost-effective to change the turbine blades more frequently than constructing a desanding basin. Such comparative analysis has been conducted during feasibility study of Sapta-Gandaki Hydroelectric Project and has been recommended the following instead of desanding basin:

i. To provide an intake wall in front of the power intake and take only surface water in which most of the large size particles are already settled.

ii. To use the 13 Chromium Hi-Nickel Steel as the material of the turbine blades. This material is durable against abrasion and the life of the turbine blades is expected to extend up to around 10 years.

iii. To repair the damaged turbine blades due to abrasion by the build-up-welding.

Inlet and Outlet Zone

Inlet Zone

The need for good inlet design cannot be overemphasized. Poor inlet design is probably the factor most responsible for "poor" basin performance. To achieve good hydraulic efficiency and effective use of the settling zone, the inlet strictly needs to distribute inflow and suspended sediment uniformly over the vertical cross-sectional area of the settling zone.

Horizontal velocity variations across the width of a desanding basin affect the hydraulic efficiency considerably more than velocity variations in depth, provided always that bed scour is avoided. Principal attention therefore needs to be given to uniform inflow distribution in the horizontal plane. Methods which are commonly adopted to achieve good flow distribution are:

- Submerged weir;
- Gradual open channel expansion, possibly using guide vanes;
- Troughs with slots or orifices in walls or bottom;
- Baffle walls; and
- Tranquiller racks

Orifices or baffled inlets are generally used only when extremely low flow through velocities is needed for water treatment. As a general rule, the inlet layout should either follow an existing proven design, or be model tested.

Outlet Zone

The operating water level of the desanding basin is generally controlled at the outlet or further downstream. If the outlet is narrower than the basin, the outlet control requires an appropriate approach transition to avoid short circuiting and to maintain an even flow distribution. The outlet contraction may be more abrupt than the inlet expansion.

Where it is practical decanting the outflow from the desanding basin over a weir is recommended. This method will generally provide a higher water quality than by the simple transition structure. The design engineer will have to decide the merits of the above approach taking into account the implications of additional construction costs and head losses for a weir take off.

Desanding Basin Hopper

Desanding basins are constructed to settle the suspended particles greater than a particular size containing in the water and to successfully flush out the settled particles through a flushing channel. Settled particles are initially stored at the bottom level of the desanding basin which is called a hopper of the basin and the hopper are constructed in such a way that its bottom width is relatively smaller than the width of the desanding basin. The slope angle of the basin and the bottom of the hopper is arranged slightly greater than the repulse angle of the sand which is usually 45^0. The narrow width at the bottom of the hopper is to create high velocity during the flushing operation to flush out all the deposited sediments in it.

3.1.1.3 Headrace Canal

General

The headrace canal serves to convey water to penstock intake, generally a forebay. In a run-of-river hydropower project, the structural components like desanding basin, cross-drainage works such as aqueducts, siphons, super passages, flumes, come across between the starting point of the canal and forebay forming the integral parts of a headrace canal. Since the silts settled in the desanding basin is to be periodically removed and discharged to the river using the higher velocity of flow, for obtaining a necessary purging head, generally this structure is located at some distance[3] from the headworks site. The headrace canal is aligned, first, to reach the site appropriate for construction of a desanding basin, then, to reach the forebay site. The first portion of this canal, connecting river intake and desanding basin, called also approach canal (Section 3.1.1.1 above) will be short to the possible extent to minimize the length of silt flushing channel, while the portion lying between desanding basin and forebay (the main headrace canal) could have a length extending up to several kilometers depending upon the head to be concentrated in a hydropower project and, therefore, will have to pass through a number of crossings. The canal needs to be aligned as contour canal in a way to minimize cut and fill. The ideal design and alignment is that in which the earth obtained from digging in the bed is equal to the earth required for the formation of banks.

Headrace canals have proven to be an economical means to alter the gradient of the river to benefit hydropower production. Although pipes and tunnels could be used to produce the same results, power canals are more economical in a favorable topographic condition. A major technical drawback is that they often require considerable maintenance to control vegetation and sediment deposition. Their costs may also be excessive if they require a substantial number of bridges / aqueducts, siphons, flumes to provide for road or stream crossings. Due to topographic constraint and geological conditions the need for covered canal in places also can not be denied. The power (headrace) canals are generally designed with side channel spillway located in one of the banks to be connected to a natural drainage by a manmade or natural channel to permit safe release of surges resulting from a sudden shutdown of the generating units. If the cost of construction of side channel spillway along with a manmade channel to connect a natural drainage is excessive, the surges in power canals may be taken care of by providing additional freeboard.

Design Consideration

The objective of the design of canal is to determine the size and configuration that meets the criteria for the least cost. The cost determination usually is not limited to construction cost alone but often includes an economic analysis of costs and benefits. The basic steps in design or analysis of canal are:

- **Established Basic Criteria:**

  a) The average velocity of flow should be such that it will generally prevent the deposition of silt or growth of weeds and avoid scour in an unlined canal. Such velocity generally lies between 0.9 and 1.8 m/s for unlined canals and for lined canal the average flow velocity depends on the type of lining.

  b) Freeboard should not be less than 0.3 m plus one-tenth of the full supply depth to accommodate fluctuations

  c) For curves along alignment, the ratio of the radius of the curve (r) to width of canal (b) should be three or greater.

  d) Side wall angles for transitions should be such that $\tan\phi$ equals one-third of the freeboard. In general, transitions should be as smooth and gradual as possible to minimize turbulence and hydraulic losses. Rounded corners are preferable to sharp edges.

[3] *"Some distance" here means a minimum distance for having a minimum level difference between the water level at the desanding basin and flood water level on the river course so that the necessary purging load is available.*

e) For not allowing the entry of rainwater from the sloppy terrain to headrace canal, the provision of catch drain in the side of upstream slopes is essential.

- **Canal Alignment**

The practical alignment is the economically shortest route. It will lie in between the routes following the contour of the side hill from dam / weir to forebay with a minimum excavation or fill and a straight line between dam / weir and forebay which would usually result in excessive cuts and fills.

- **Basic Geometry**

The design of a power canal as with any design problem, aims at determining the size and configuration that meets the criteria for the least cost. The best form of cross-section of a canal is a section which gives maximum discharge for a minimum cross-section for a given bed slope. The cross-section should also correspond to the section with the least loss of water from absorption (i.e. with minimum of wetted perimeter) and evaporation (considered only if the canal is very big in size and alignment is located in very hot and low humid area). Theoretically, a circle for a closed canal, a semi-circle for open canal, half a square (i.e. depth equal to half the width) for rectangular canal and semi-hexagon for trapezoid canal are the best discharging canals. In practice for ease of construction, cross-sections close to the theoretical one will have to be adopted. Depending upon the location, humidity and size of the canal, an evaporation loss is considered*.

In the Nepalese conditions, the derivation type of hydropower projects have prospects in the mountainous regions, because in the Terai region very little head could be concentrated at a given location even with very long water conveyance system. The terrain conditions in the mountainous regions are such that there are topographical limitations for constructing wide, but shallow canals. On the other hand for irrigation development which has larger prospect in the Terai, due to flatness of topography, wide and shallow canals will have more prospect. Thus, the topographical conditions could be considered as the major criteria for determining whether the canal should be shallow or deep.

The most efficient proportions of the rectangular, trapezoidal and parabolic canal are:

<u>*For rectangular Canal*</u>

$$b = 2h \qquad \text{---} \qquad (3.3)$$

$$R = A / P = h / 2 \qquad \text{---} \qquad (3.4)$$

For the case of section that is relatively wide in comparison with depth (Width>25 x depth),

$R = h$, more generally the value of R is between h and $h/2$.

<u>*For trapezoidal canal*</u>

$$\frac{b}{h} = 2\left(\sqrt{1+z^2} - z\right) \qquad \text{---} \qquad (3.5)$$

Where,

b	=	Width of the canal
h	=	Depth of the canal
z	=	Side slopes
R	=	Hydraulic radius
P	=	Wetted perimeter, and
A	=	Area

* *Please refer "Hydrology in Practice" Elizabeth M. Shaw – Third Edition – Stanley Thornes Publishers Ltd. United Kingdom, for the calculation of absorption and Evaporation losses in headrace canal.*

The most efficient hydraulic section is when the top width is twice the length of the sloping side.

For Parabolic Canal

$$T = 2\sqrt{2}h \qquad \text{---} \qquad (3.6)$$

where,

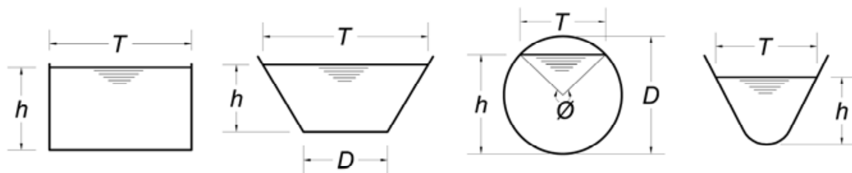
T = Top width

h = Depth

Geometric Elements of Channel Sections

Unlike the natural channels which are usually irregular in shape, man-made channels (artificial channels) are usually designed with sections of regular geometric shape. Some of the commonly used geometric sections and corresponding formulas are given below in the tabular form.

Geometric Sections and Corresponding Formulae



Formula/Shape	**Rectangular**	**Trapezoidal** (“z”-Side Slope)	**Circular**	**Parabolic** T-Top width
Area-“A”	b *h	$(b+zh)h$	$\frac{1}{8}(\phi - \sin\phi)D^2$	$\frac{2}{3}Th$
Wetter Perimeter-“P”	b+2h	$b + 2h\sqrt{1+z^2}$	$\frac{1}{2}\phi D$	$b + \frac{8h^2}{3T}$
Top Width of Section-“T”	b = T	b+2zh	$2\sqrt{h(D-h)}$	T
Hydraulic Radius-“R”	$\frac{bh}{b+2h}$	$\frac{(b+zh)h}{b+2h\sqrt{1+z^2}}$	$\frac{1}{4}\left(1 - \frac{\sin\phi}{\phi}\right)D$	$\frac{2T^2h}{3T^2+8h^2}$
Hydraulic Depth-“D”	h	$\frac{(b+zh)h}{b+2zh}$	$\frac{1}{8}\left(\frac{\phi - \sin\phi}{sen\frac{\phi}{2}}\right)D$	$\frac{2}{3}h$
Section Factor	$bh^{1.5}$	$\frac{[(b+zh)h]^{1.5}}{\sqrt{b+2zh}}$	$\frac{\sqrt{2}(\phi - \sin\phi)^{1.5}}{32\sqrt{\sin\frac{1}{2}\phi}}D^{2.5}$	$\frac{2}{9}\sqrt{6}Th^{1.5}$

The typical sections of the lined and unlined canals are shown in the Figure 3.3 below.

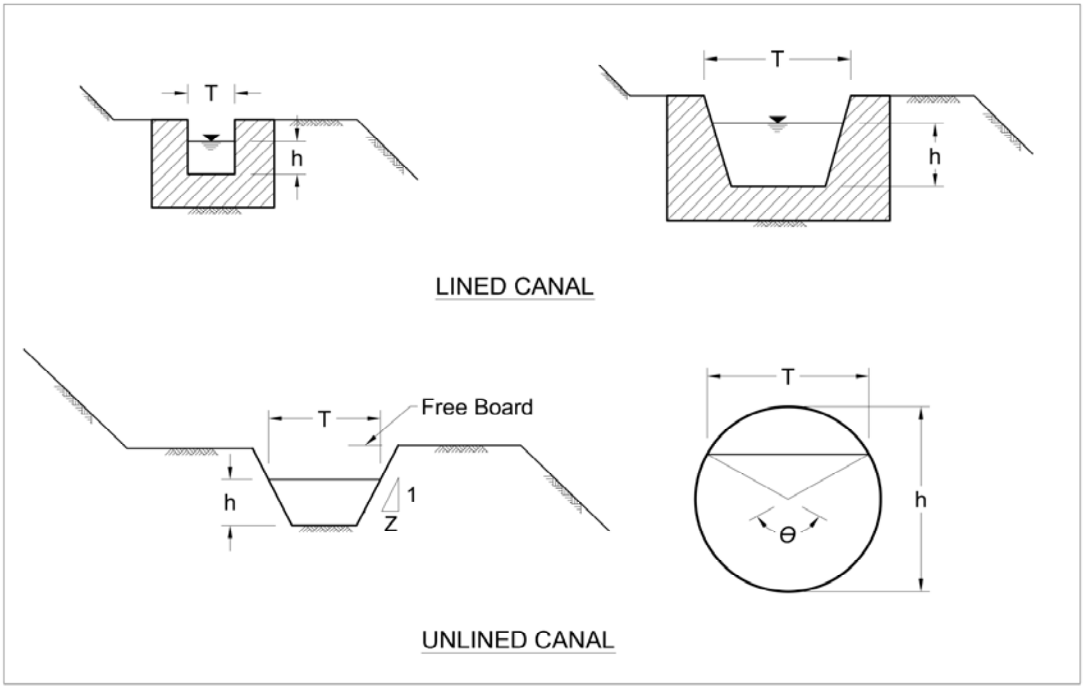
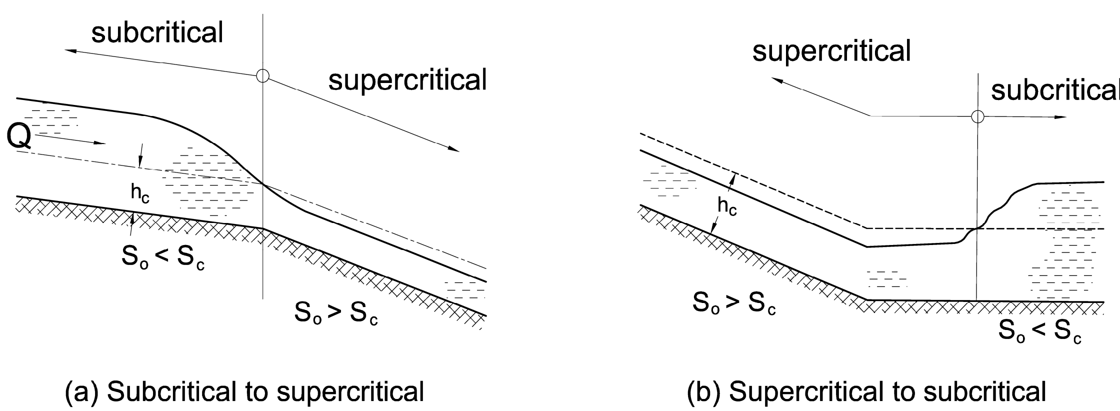


Figure 3.3: Typical Sections of the Canal

- **Design for Transition**

Transitions are required to alter the basic canal geometry. Sidewall angles for transitions should follow the basic criteria outlined previously. It is important to check if the transition requires a change in state of flow, from sub-critical to super-critical or vice versa. Special design considerations like energy dissipation systems are required at such change. In general, transitions should be smooth and gradual as possible to minimize turbulence and hydraulic losses. Rounded corners are preferable to sharp edges.

Hydraulic Transition –Transition between sub-and Super-Critical flow: If sub-critical flow exists in a channel of a mild slope and this channel meets with a steep channel in which the normal depth is super- critical there must be some change of surface level between the two. In this situation the surface changes gradually between the two. The flow in the joining of the two channels the depth passes through the critical depth (Fig. 3.4a).



h_c = critical depth
S_c = critical slope
S_o = canal slope

Figure 3.4: Transition of Sub to Super-Critical Flow and Vice Versa

If the situation reversed and the upstream slope is steep (super critical flow) and the downstream mild (sub-critical), then there must occur a hydraulic jump to join two (Fig. 3.4b). Also, there may occur a short length of gradually varied flow between the channel junction and the jump.

Structural Transition: The terminology transition structure implies a designed channel appurtenance whose purpose is to change the cross sectional shape of the channel. The function of such structures is to:

- Avoid excessive losses of energy
- Eliminate cross waves, standing waves, and other turbulences
- Provide safety for both the transition structures and the waterway.

The geometric form of a transition structure can vary from a rather simple straight line design to a complex, streamlined design involving warped surface. The common types of transitions are generally used at the inlet and outlet of structures and where changes occur in the water section. An accelerating water velocity usually occurs in the inlet transitions and a decelerating velocity in outlet transitions. The most common type of open transitions where a change of section from the trapezoidal canal to the rectangular opening of structures occurs, are the streamlined warp, straight warp, broken back and vertical walled type.

There are considerable difficulties in constructing those sloping parts of the sloping warped which are steeper than the natural slope of the retained materials, but not so steep that the lining can stand unaided to allow backfilling behind. Accordingly, in Nepal either broken back “dog leg” or a vertical walled transition should be used. Hydraulically broken back transition is the better solution. In a vertical walled solution, to reduce head losses to a minimum the wing walls are flared at 45^o to the canal centre line and lead into the abutments via a horizontal circular arc.

For minimum hydraulic loss and smooth operation, a small submergence of the opening in the headwall should be provided at inlet transition, and no submergence of the opening in the headwall should normally be provided at outlet transitions. If the submergence exceeds one sixth of the depth of the opening at the outlet, the hydraulic loss should be computed on the basis of the sudden enlargement rather than as an outlet transition. The hydraulic loss in a transition will depend primarily on the difference between the velocity heads at the open end of the transition and at the normal centerline section of the closed conduit at the headwall. Hydraulic loss coefficients in some transitions are tabulated below:

Type of open transition to closed conduit	Inlet	Outlet
• Streamline warp to rectangular opening	0.1	0.2
• Straight warp to rectangular opening	0.2	0.3
• Straight warp with bottom corner fillets to pipe opening	0.3	0.4
• Broken back to rectangular opening		
• Broken back to pipe opening (closed)	0.3	0.5
• From Trapezoidal canal to rectangular opening through vertical walled transition	0.4	0.7
• From trapezoidal canal to pipe opening through vertical walled transition	0.5	0.8
	0.6	1.0
Closed Transition		
• Square or rectangular to round (Maximum angle with centerline =7.5^0)	0.4	0.7

Open transitions to multiple closed conduits will involve some additional hydraulic loss. Average friction loss should be added for large transitions, but it may be neglected for small transitions. The slope of the floor on the broken back outlet transitions should be 1:6 or flatter. The maximum angle between the water surface and the centerline should not exceed 27.5^0 for inlet transitions and 22.5^0 for outlet transitions for the best hydraulic conditions. In some structures it may prove economical to use 25^0 to allow the same structure to be used for both inlets and outlets. A 30^0 angle is often used on inlet transitions with checks, in which case an additional loss is allowed for the check. Design should provide for a loss through most check structures of about 0.5 times the difference in velocity head through the check opening and the upstream channel section.

Transmission sections of canal are normally designed for minimum head loss. The basic two methods are generally adopted as a formula for the transition (refer the figure given below):

$$B_x = \frac{B_c \times B_f \times L_f}{L_f \times B_c - (B_c - B_f) \times x} \qquad \text{-------------------------------} \quad (3.7)$$

and

$$x = \frac{L_f \times B_c^{\frac{3}{2}}}{B_c^{\frac{3}{2}} - B_f^{\frac{3}{2}}} \left\{ 1 - \left(\frac{B_f}{B_x} \right)^{\frac{3}{2}} \right\} \qquad \text{-------------------------------} \quad (3.8)$$

where

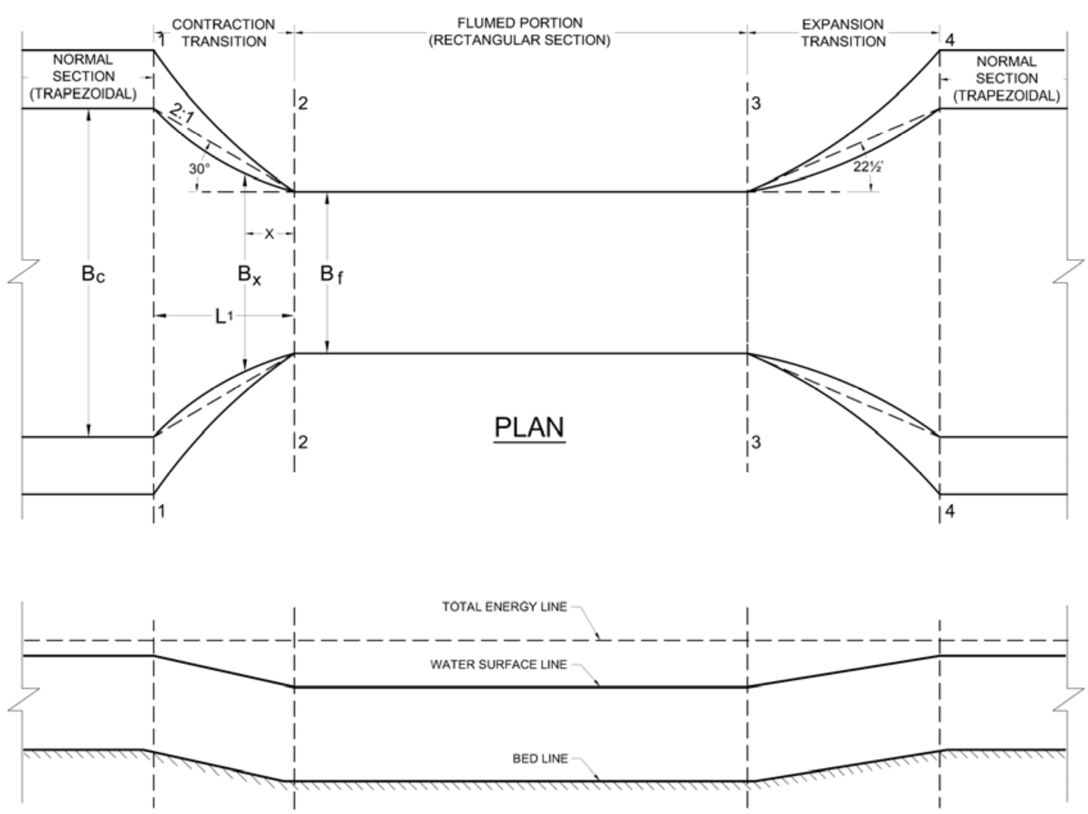
B_x = Width of transition

B_c = Width of the normal canal section

B_f = Width of the flamed section

L_f = Total length of the transition

x = Distance of transition



L-SECTION

An inlet transition connects to a free flow closed conduit in such a way that the conduit inlet is sealed the quantity of water that is passed should be determined by the orifice equation. The head should be measured from the center of the opening to the inlet water surface and an orifice coefficient of c=0.6 should be used. A small correction factor is theoretically required when the submergence is less than the height of the opening. When the inlet to a long conduit may operate without sealing, a hydraulic jump may occur that can result in blowback and undesirable operation. Transitions to free flow conduits can have the control point anywhere between the inlet cutoff and the headwall. If control at any flow is at the inlet cutoff, the upstream channel must be protected from erosion or the design changed to move the point of control to the transition.*

- **Head Losses due to Friction and Structural Transition**

Water flowing through a conveyance system with entrances, bends, sudden contraction and enlargements, racks, valves and other (in case of pipe) experiences, in addition to the friction loss, a loss due to the inner viscosity. This loss depends on the velocity and is expressed by an experimental coefficient *K* multiplying the kinetic energy $v^2/2g$.

Calculation of head losses is based upon the following general equation.

$$H_L = h_f + h_t \quad \text{---} \quad (3.9)$$

where,

H_L = the total head loss or energy loss;
h_f = the loss due to frictional resistance, and
h_t = the loss due to transitions or changes in direction, also called local losses.

The friction loss (h_f) in length is calculated by the equation:

$$h_f = \frac{n^2 * V^2 * L}{R^{4/3}} \quad \text{---} \quad (3.10)$$

where,

h_f = friction loss
V = velocity in m/s.
R = hydraulic radius in m,
n = Manning's roughness coefficient, the typical values of which are given in Table-3.1#, and
L = length of canal in m.

* *For more detailed information about the structural transition refer to standard hydraulic books such as (Water Power Development – Emil Masonyi, Hydraulic Design – Ven Te Chow, Hydraulic Design – French, Davi's Handbook of Applied Hydraulics – ISBN 0-07-073002-4).*

\# *For more Manning's roughness value –refer to standard hydraulic books such as (Water Power Development – Emil Masonyi, Hydraulic Design – Ven Te Chow, Hydraulic Design – French, Davi's Handbook of Applied Hydraulics – ISBN 0-07-073002-4).*

Table-3.1: Typical Manning's Roughness Coefficients for Channels

Channel Material or Type	Manning's n
Clean, straight earthen channel	0.022
Straight earthen channel with grass	0.027
Winding and sluggish earthen channel with some weeds	0.030
Winding and sluggish earthen channel with cobble bottom and clean sides	0.040
Unmaintained earthen channel with uncut weeds and brush on sides and clean bottom	0.050
Concrete-lined channels	0.013 - 0.017
Asphalt-lined channels	0.013 - 0.016
Gunited channels	0.019 - 0.022
Rubble masonry	0.020 - 0.025
Riprap-lined channel	0.030
Channel with cement plaster	0.011
Brick work	0.014
Rock cut channel	0.035 - 0.040
Channel with gravel	0.022 - 0.030
Older wooden channel	0.015
Natural river bed	0.024 – 0.05

Transition losses (h_t) and local losses such as entrance / exit loss, trash rack loss, bend loss, etc. is calculated by the following general equation:

$$h_t = \frac{KV^2}{2g} \quad \text{-- (3.11)}$$

Where, K is loss coefficient, and g = gravitational constant.

The value of *K* depends on nature of transition (expansion / contraction), change in direction (shape and angle of bend) and resistance to flow imposed. For other details of head loss calculation and loss coefficient refer Annex-2.

Maximum Permissible Velocity in the canal

The maximum permissible velocity or the maximum allowable bottom shear stress in the power canals will be limited by the resistance of the bed material to erosion or, in case of lined canals, by that of the lining against wear. The latter becomes considerable if the water carries abrasive materials in appreciable quantities.

Some authors relate permissible bottom velocities to the material of bottom and sides or /and lining, while others suggest values for the permissible mean velocity. Maximum permissible velocity have been determined partly by experimental and partly by theoretical research work.

The maximum bottom velocity, i.e. critical scour velocity as regards erosion given by Sternberg is:

$$V_b = \xi \times \sqrt{d} \quad \text{-- (3.12)}$$

Where,

V_b - Maximum Permissible Velocity

d - Diameter of Particles in m. and in this case ξ =4.43

Although a number of other investigators (Kutter, Airy, Hochenberger, Schffernak, Mavis and others) have derived theoretical relations between critical bottom velocity and particles size, the discussion on these relations are not indicated. The practice refuses calculation with the bottom velocity, because its deduction from the data available for designing is uncertain; it would impose a rather difficult task on the designer. In fact it is not yet unambiguously clarified what is meant by bottom velocity. For this reason two other types of relations are commonly used in engineering practice to define the scouring effect of the flow: the mean velocity or the bottom shear stress versus the particle size. Since the incipient motion of the bed material strongly depends on the intensity of turbulence, the present trend is to take into account this factor by indicating the shear velocity and expressing the critical state as dependence of both parameters; the shear stress and the shear velocity. Since the specified gravity of the grains is of great significance, methods and expressions, which include the influence of this property of the material, can cover a wider range of the erosion phenomena. Accordingly, in several cases, priority can be given to the procedures which relate the incipient scouring to the specific (dimensionless) shear stress:

$$\tau_* = \frac{\tau}{(\gamma_1 - \gamma) * d} \quad \text{---} \quad (3.13)$$

Where,

τ_* - Specific shear stress

$\tau = \gamma RS$ = Bottom Shear Stress in Kg/m^2

γ_1, γ - Specific weight of particles and the water respectively in Kg/m^3

d - Mean or representative grain size (diameter) in meter.

R - Hydraulic Radius

S - Slope

Scour velocities for various soil particles sizes based on the American practice are indicated in the figure (Fig. 3.5) below: This figure is created by *W.P. Creager* and *J.D. Justin*.

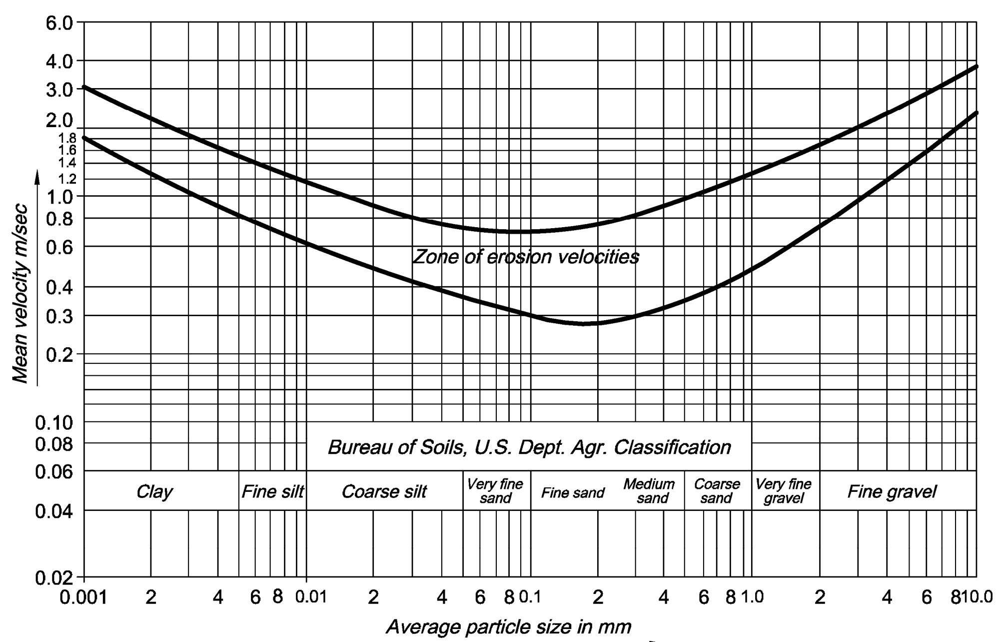


Fig. 3.5: Scour Velocities for Various Soil Particles Sizes

The range of maximum permissible mean flow velocities is given for different soil grain diameters varying from fine clay to gravel of medium fineness (0.001 to 10 mm). The erosion velocities depend on the number of soil properties, beside the average particle size. Safe values are characterized in American practice by a fairly wide range instead of a single curve. Some maximum permissible mean velocities are given below in the tabular form (Table 3.2)[4]:

The Table 3.2 below has been compiled for the loose granular bed material on the basis of velocity distribution pertaining to a depth of 1 meter. In case of depths other than that, corrections are to be introduced. With water depths lower than 1m, tabulated permissible velocities are to be diminished

[4] *Reference from Low Head Power Plant by Emil Mosonyi*

because of the more uniform velocity distribution and vice versa. Actual permissible maximum mean velocities will be obtained from tabulated values V_1 (Table 3.3) as:

$$V = \alpha V_1 \qquad \text{(3.14)}$$

Where,
V - Actual permissible maximum mean velocity
V_1 - Maximum Mean Velocity
α - Coefficient whose value depends on the depth

The maximum permissible velocities for solid rocks are given in Table 3.4 and the same for cohesive soils and for flow in lined canals are given respectively in Tables 3.5 and 3.6.

Table 3.2: Maximum permissible mean velocities for loose granular bed material

Material	**Diameter of particle *"d"* in mm**	**Maximum mean velocity in case of h=1 m , V_1 m/s**
Very coarse gravel	200-150	3.9-3.3
	150-100	3.3-2.7
	100-75	2.7-2.4
Coarse gravel	75-50	2.4-1.9
	50-25	1.9-1.4
	25-15	1.4-1.2
	15-10	1.2-1.0
	10-5.0	1.0-0.8
	5.0-2.0	0.8-0.6
	2.0-0.5	0.6-0.4
Cobble	0.5-0.1	0.4-0.25
Coarse sand	0.1-0.02	0.25-0.20
	0.02-0.002	0.2-0.15

Table 3.3: Correction coefficients to formula $V = \alpha V_1$

Depth "h"	Correction coefficient α
0.3m	0.80
0.6m	0.90
1.0m	1.00
1.5m	1.10
2.0m	1.15
2.5m	1.20
3.0m	1.25

Table 3.4: Maximum Permissible velocities for solid rocks

Material	**V_1, m/s**
• Loose conglomerate, clay loam	2.5-3.0
• Tough conglomerate, porous lime rock, stratified limestone	3.0-5.0
• Dolomite sandstone, non stratified limestone, quartzitic limestone	4.5-7.0
• Marble, granite, syenite, gabbro-coarse	15.0-25.0
• Same as above - smoothed	27.0-38.0
• Porphyry, phonolite, andesite, diabas, basalt, quartzite - coarse	24.0-48.0
• Same as above - smoothed	38.0-45.0

Table 3.5: Maximum permissible mean velocities for cohesive soils

Type of Soil	V_1, m/s	Note
Slightly clayey sand, very fine sand	0.7-0.8	Tabulated data apply to hydraulic radius between 1 and 3 m for R>3 increased by $(R/3)^{0.1}$
Compacted clayey sand	1.0	
Loose sandy clay or loess	0.7-0.8	
Medium sandy clay	1.0	
Hard sandy clay	1.1-1.2	
Soft clay	0.7	
Ordinary clay	1.2-1.4	
Rolled clay	1.5-1.8	
Silts	0.5-0.6	

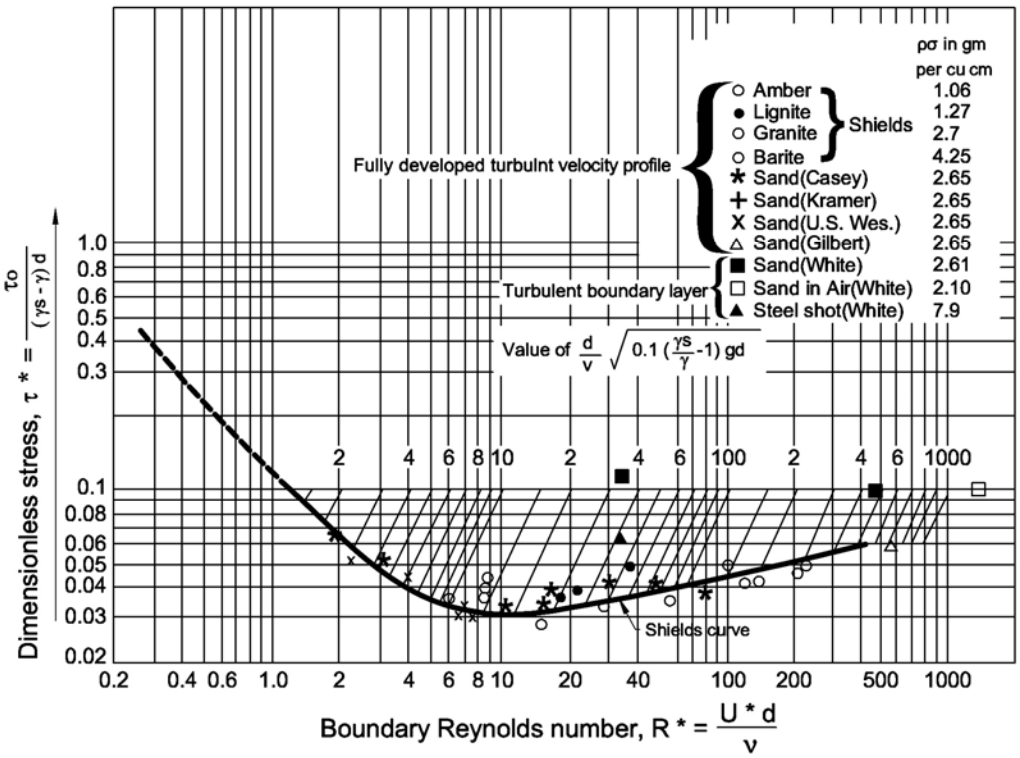
The accuracy obtained by the use of values listed above is limited and sufficient only for the design of projects of smaller significance, or for the preliminary study of greater ones. However for a detailed planning of the latter, a more exact investigation of non eroding mean velocity is advisable.

Table 3.6: Maximum permissible mean velocity for flow in lined canals

Type and Strength of lining	Permissible velocity, V, m/s
Brick (crushing strength of water 16-30 kg/cm^2)	1.4
Soft, sedimentary stone lining	2.4
Clinker (crushing strength 120 kg/cm^2)	5.8
Timber lining	6.0
Concrete having a 28 day cube strength of 210 kg/cm^2	7.4
170 kg/cm^2	6.6
130 kg/cm^2	5.8
110 kg/cm^2	4.4
90 kg/cm^2	3.8

Maximum Permissible Shear Stresses:

This method of determining the bed erosion is based on the establishment of the critical bottom shear stress versus the shear velocity, investigated firstly by Shields, who experimentally determined the dividing line between states of motion and no motion of the particles in given condition. Shields combined expressions for the destabilizing forces drag and lift against weight or friction as the stabilizing force into a general formula for the equilibrium of particles and are given below in a diagram form:



On this diagram the specific (dimensionless) shear stress is plotted versus the Reynolds number related to the shear velocity and the grain size (termed as Sear Reynolds number).

$$R* = u * d/\nu \qquad \text{--} \qquad (3.15)$$

Where, the shear velocity is expressed as

$$u* = \sqrt{\tau/\rho} = \sqrt{gRS} \cong \sqrt{ghS} \text{ m/s}$$

Here, ρ denotes the density and ν the kinetic viscosity of the water. The ν strongly varies with the temperature. Shear Reynolds number is represented by $R*$ and the shear velocity by $u*$. Then the above equations takes the shape of:

$$\tau_* = \frac{u*^2}{(\gamma_1 - \gamma)gd} \qquad \text{--} \qquad (3.16)$$

This expression can be defined as the Froude number related to the submerged grain.

Lowest Permissible Velocities

In order to prevent settling of fine particles carried as suspended load by the flowing water i.e. to avoid silting of the canal, lowest permissible velocities should be determined. This problem can be solved by two approaches: the criterion for non-silting may be related to the lowest permissible mean velocity or to the suspended load carrying capacity of the flow.

According to A. Ludin no sedimentations is likely to occur if the mean velocity
V > 0.3 m/s in case of silty water and
V> 0.3-0.5 m/s in case of water carrying fine sand.

For the determination of non-scouring, non-silting velocity in irrigation canals in India, R.C. Kennedy proposed the following formula:

$$V = ch^{0.64} \text{ m/s} \quad \text{---} \quad (3.17)$$

Where,

V - Minimum permissible velocity

h - depth of the water in meter,

c - coefficient whose value varies from 0.54 to 0.70 according to the character of silt load. This relationship is based on the extensive research work.

For the appropriate values of minimum permissible mean velocity M.M. Grisin suggested the formula:

$$V = AQ^{0.2} \text{ m/s} \quad \text{---} \quad (3.18)$$

with the following value of coefficient A respectively,

w……………………………<1.5 1.5-3.5 >3.5 mm/sec
A………………………….. 0.33 0.44 and 0.55

where w is the settling velocity and Q is the discharge in the canal.

For calculating the lowest permissible velocity using silt carrying capacity, several expressions are derived by number of authors. But the old empirical formula which does not consider the suspended load in the canal should not be regarded as reliable in every instance as they do not allow for the factors that affect the movement of suspended silt. One of the recent trends, therefore, is to conduct investigations on the basis of the silt load carrying capacity of canals rather than suggesting expressions for velocity limits. This is exemplified by the formula of E.A. Zamarin for unlined canals:

$$G_0 = 700\frac{V}{w_0}\sqrt{\frac{RSV}{w}} \quad \text{Kg/m}^3 \quad \text{--------------------} \quad (3.19)$$

where,

G_o = silt load carrying capacity of water in kg/m^3

V = mean velocity of the flow in m/s

R = Hydraulic radius in m

S = gradient of the water surface

w = The mean settling velocity (in still water) in mm/sec.

w_o = w if w>2mm/sec but

w_o = 2mm/s, if w$\leq$ 2mm/s

Informative value of sufficient accuracy can be obtained by the simplified formula:

$$G_0 = \frac{9000}{w_0 - 1.25} RS \quad \text{Kg/m}^3 \quad \text{------------------------------} \quad (3.20)$$

if the silt charge in the water is smaller than the silt load carrying capacity of canal the expression will be $G<G_o$.

Free Board with respect to the Lined and Unlined Canal

Free board of the canal should be 0.3-0.5 m above the possible surge in the canal but substantially greater allowance may also be required. Freeboard may be determined, furthermore, by the height of waves due to wind action, in which case prevailing wind direction and the width of the canal will be deciding factor or the surge caused by the interruption in power turbine might be the deciding factor for the freeboard height. There is no universally accepted rule for the determination of freeboard since wave action or water surface fluctuation in the canal may be created by many uncontrollable causes. The designers should, however, consider the following:

- Pronounced waves and water surface fluctuation can be expected in canal with velocities and slopes steep enough to cause unstable flow condition
- Curve with high deflection angles where water traveling at high velocities can cause significant super elevated water surface on the convex side of the canal.
- In the canal where velocities approach critical levels the water may flow at alternate depths and jump from the low stage to the high stage when an obstruction is encountered.
- Siltation or blockage due to landslides can cause significant changes to the capacity of the canal section
- Drainage inflows, from intense, short duration storm can cause a wave or tidal effect
- Sudden closure of the canal by the turbine operator (tripping of the power station) will also cause a tidal or backwater effect and a significant change in water surface elevation.

The freeboard for an unlined canal should take into account deterioration of the bank, which is generally common in Nepal. Freeboard can be treated as a function of canal depth and flow and the total freeboard is, therefore, related to canal capacity. Suggested values of freeboard in relation to the lined and unlined canal capacity are given in Table 3.7.

Table 3.7: Freeboard in unlined and lined canals

Canal discharge in m^3/s	Unlined canal in meter	Lined canal in meter
Upto-0.10	0.20	0.10
0.1-0.5	0.30	0.15
0.5-1.0	0.40	0.15
1.0-1.5	0.45	0.20
1.5-2.0	0.50	0.20
2.0-3.0	0.55	0.25
3.0-5.0	0.60	0.30
5.0-10.0	0.70	0.35
10.0-30.0	0.80	0.45
Above 30.0	1.00	0.50

Reference: Irrigation Design Manuals for Irrigation Projects in Nepal, PDSP, 1990

Freeboard can vary from less than 5% to more than 30% of the depth of flow depending on the designer's estimation of the probable changes in water surface elevation.

Catch Drainage

Catch drains are designed to take the runoff from the drainage area resulting from a storm which is considered to produce the greatest momentary runoff that can damage the main headrace canal system in the hydropower project/s. Estimation of excess storm runoff is necessary in order to design a field surface drains which will shed it without erosion and to design disposal by channel having adequate capacity. Generally, when rain falls on a dry catchments a part is intercepted by the vegetation cover, part infiltrates into the soil and part fills the depressions in the ground before overland flow begins. For the single period of rain and a given type of cover interception can be taken as a fixed depth of water but infiltration proceeds at a rate which falls with time in a manner dependent on the soil and its internal drainage. The proportion of rainfall which becomes surface runoff, therefore varies with the intensity and duration of precipitation and also with the initial moisture content of the soil. Frequency, intensity and duration of storms are related by statistical analysis. Frequency is the average number of times an event occurs in a given number of years. Recurrence interval is the average number of years between occurrences of that event. Design storm for drainage works is that for which the recurrence interval is acceptable. In general an interval of 10 years of return period flood / runoff is considered as a design parameter for the drainage design in both hilly and Terai area of Nepal.

- **Optimization Study**

Optimized design of canals must consider the flowing cost factors:

- Construction cost
- Maintenance cost
- Operation cost
- Value of power and energy due to hydraulic head losses
- Life time of the canal will be considered 50 years for the economical analysis and 25 years for the financial analysis

Economic analysis may be performed on a present worth or annual cost basis. The design objective is thus to obtain the configuration which minimizes the total cost. Several trail design are analyzed and a curve is to be developed to select the most economical design.

- **Lining**

Hydro-electric canals are now usually provided with some form of lining, which permits high velocities, greatly reduces the loss by leakage and gives a more favorable value of Manning's n. The design discharge can therefore be carried by a smaller canal and the cost of the lining is generally offset by the resulting economics in excavation and the reduction of maintenance costs and of leakage. In the case of headrace canal, there is also possibility that the lined canal will have the flatter slope, and will permit the development of greater head. However, before going to any type of lining, it is recommended to have alternative study of lined and unlined canal.

Power canals are lined for one or more of the following reasons:

i. Reduction of the roughness coefficient: The slope necessary for the conveyance of a given discharge at a given cross section is flatter, or else the discharge conveyed at a given slope is greater in a lined canal than in a similar earth canal.
ii. Increase of the permissible velocity: In order to reduce the cross section of a canal, the permissible velocity in a lined canal may be increased several times up to a certain limit without using a steeper slope, and above that – by increasing the slope as well. Lining protects the canal against erosion at a flow of greater velocity.
iii. Reduction and or prevention of seepage discharge
iv. Increasing the slope of the banks which reduces the area occupied by the canal. A steeper slope is also a better hydraulic solution.
v. Partial lining around the water surface for protection against wave action. To determine the width of the lining, the fluctuation in the operational water level and the slope of the sides must be known. The safety margin of the lining above the highest and below the lowest operational water level and the slope of the sides must be known. The safety margin of the lining above the highest and below the lowest operational water level is governed by the width of the water surface, the depth of the canal and the prevailing direction of wind.

In certain situations, the available materials along the bed and banks of canals may be too pervious resulting in significant seepage losses. In these cases, the canal bed and banks have to be lined with impermeable materials. In other situations, the topography along the canal alignment and cost of excavation may require that the canal be designed with a relatively steep bed slope resulting in higher velocities of flow than permissible non-scouring velocities for earthen canals. In these cases the canal bed and banks have to be lined with erosion-resistant materials. The canal lining will be economically justified if the cost of providing canal lining is less than the value of the benefits resulting for it.

By lining a canal following benefits could be derived:

i. Saves in water losses due to seepage;
ii. Prevents water logging and efflorescence of adjacent lands due to seepage;

iii. Reduces the canal section considerably and saves the costs of excavation and land acquisition;

iv. Reduces the Maintenance cost;

v. Prevents the growth of weeds;

vi. Reduces bank erosion and breaches;

vii. Also, prevents water absorbing salts while passing through Kalarish tracts; and

viii. Filtering of water by cultivations is diminished.

But these savings are to be compared with the costs involved in lining.

Various types of lining are used. They have their own merits and demerits. Use of types of lining depends also on the materials available at work site. The most commonly used are described below:

i. *Stone paving, without cementing materials*: These types of canal lining are mostly stone paving without cementing materials as riprap, simple paving, pavement upon a gravel or crushed stone drainage layer and rock fill are also used for protection against erosion. But these types of linings do not reduce seepage or friction. Maximum allowable velocities with these linings are about 2 m/s. Thickness of the lining depends on the velocity of the flow in the canal (velocity of the flow should not wash away the paved stones). This will be calculated using formula of maximum permissible shear stress.

ii. *Clay Puddle*: It is suitable only for the canals with perennial flows as it is liable to develop cracks on drying. Puddle lining is quite satisfactory, because it reduces seepage by about 80%, but can only be used if good clay is available. A layer of 7.5 cm to 15 cm is spread on the bed and side slopes and covered with 25 to 30 cm of silt. Permeability of soil can be considerably reduced by treating it with bituminous material. The technique consists in adding the bituminous mixture to the ordinary mud plaster and then applied to the channel in thickness of about 12 mm which should be allowed to dry.

iii. *Brick and Stone Lining in Cement Mortar*: Brick and stone lining have the advantage that no expansion or contraction cracks are formed as with concrete lining. But in case of the bricks, they should be sandwiched with a layer of cement mortar, as bricks are porous. Brick lining in particular gives a saving of about 75% in losses of water by absorption and percolation as compared with the earthen canal section. Bricks are laid either in single layers flat, or on edge or flat in two layers. The bricks are usually laid in herringbone pattern and are bedded on 12 mm layer of 1:5 cement mortar laid on the consolidated and damped soil in 1:3 cement mortar. A layer of 12 mm cement plaster 1:3 is sandwiched in between the two layers of bricks. In floor of canal works the top brick on edge should be laid diagonally to the centre line of the channel or in herringbone pattern. The lining may be plastered of pointed on the face and may also be reinforced. The minimum thickness of this type of lining equal to the brick thickness which is 5.7 cm in case of locally available bricks in Nepal.

iv. *Soil Cement Lining*: Stabilized soil with 5% of cement is to be compacted in a 75 mm layer and topped with 6 to 12 mm thick cement sand plaster. Soil cement lining consists of a layer of mixture of cement and soil with certain moisture content laid on the sub-grade. There are two types of soil cement lining: compacted and plastic. The compacted soil cement lining is also known as standard soil cement lining in which the soil cement mixture is compacted to its maximum density keeping the moisture content of the mix at above optimum value. The plastic soil cement lining is laid with soil cement mixture in plastic form, this type of lining has higher water and cement content and a consistency similar to that of masonary mortar. These properties permits placement of plastic soil cement lining by means of a slip form similar to that used in the placement of cement concrete lining. The usual thickness of the lining is 75 to 100 mm. Further for compacted soil cement lining the quantity of the cement may vary from 5 to 10 percent by weight of the dry soil. On the other hand for plastic soil cement lining the quantity of cement may vary from 10 to 20 percent by weight of the dry soil.

v. *Concrete Lining*: In power canals, concrete linings are extensively used because they meet three basic requirements - they engender little friction, protect against erosion and reduce / prevent seepage. The two types of commonly used concrete linings are concrete poured in-situ and the prefabricated slabs. For abrasion resistance lining, particularly in the approach canal, steel lining could be an option.

The thickness of the concrete lining is fixed depending upon the nature of channel required, namely, hydropower channel or irrigation channel, full supply depth and channel capacities. Hydel channels have a greater thickness than the channels used for other purposes because of the drawdown effects and where closure of repairs may not be usual. Deep channels have greater thickness than shallow channels. Minimum thickness of channel lining based on the capacities of the channel are given below:

Capacity of channel (m^3/s)	**Depth of water (m)**	**Thickness of lining (mm)**
0-5	0-1	50-60
5-50	1-2.5	60-70
50-200	2.5-4.5	75-100
200-300	4.5-6.5	90-100
300-700	6.5-9.0	120-150

A variation of +/- 10 mm in the thickness of the concrete lining is allowed provided average thickness is not less than the specified thickness.

vi. *Reinforced Concrete Lining*: This type of lining is constructed if the relatively wide control joints are to be avoided. This type of lining will also be advantageous where great fluctuation in temperature occurs. This type of canal engenders little friction, protects against erosion and reduces seepage. The minimum thickness of the reinforcement concrete is also similar to the concrete thickness mentioned above with an additional layer of reinforcement.

vii. *Lining made of lean mixture and stone paving:* This type of lining is sprayed with cement mortar (gunite process) to reduce seepage. This will even smooth the surface of rough- e.g. stone-paving etc. the sprayed lining is preferably made of one, two, three layer, each of a thickness of 0.5 to 1.5 cm with a time lag of two or three days between the individual layer.

A 2.5 to 5.0 cm thick layer of sprayed concrete with a wire mesh reinforcement can also be used as an independent protective coating on granular soil (gravel, sand).

viii. *Bitumin and Asphalt linings, plastic membranes linings* are also used. Bitumen and asphalt linings also proved to be very efficient, and up to the present, have been applied mainly in irrigation canals. They are more water tight and elastic than concrete linings and resist rather well to atmospherical influences. The thickness of asphalt linings should range from 3 to 6 cm. In order to improve strength, reinforcement can be applied if necessary. Since roots easily penetrate through asphalt linings, herbicides should be applied before pouring. For the last type of lining the detailed information could be had from the material manufacturers.

ix. *Bentonite lining:* a thin layer of dry bentonite is first spread up to the trimmed slope, followed by a mixture of bentonite and wet earth and subsequently compacted. According to B. Bauzil a lining of 5 to 8 cm thick protected by a sand or gravel layer of 10 to 20 cm thickness, will generally meet the requirements.

x. *Pointing as an option of lining:* Pointing of the canal is usually considered in brick and stone lining in cement mortar. This method is adopted to tighten the bond between the brick or stones and prevent seepage. In general the type of lining is not in practice in hydropower canals.

xi. *Provision of weep holes, side drains and flap valves*: Weep holes are provided at the side walls of the canal to reduce the water pressure on the canal side walls from the ground behind. The weep holes will allow the water to pass through it and hence release hydrostatic pressure.

Flap valves are often called one way valve. They are usually provided on the side walls and on bottom slab of the canal to release the hydrostatic pressure behind the walls and floor. The flap valves will automatically open when hydrostatic pressure is greater than the allowable value. Side drains are provided to collect the safely bypassing of the runoff flow on the sides of the canal. More explanation of the side drain is provided in catch drain.

The following points are recommended for general considerations during lining of the canals.

i. Forms of cross-section: Since a lined canal allow a higher velocity it would seem more economical to use narrow and deep sections with sides at the same slope as the angle of response of the soil and a circular bed;

ii. In deep canals life saving devices should be provided which may consist of steps or flat slopes at intervals for cattle and some ladders for men;

iii. With a view to avoid earth pressure against the lining, the sides of the canal to be lined should preferably keep at the natural slope of the soil. Where the side slopes are made steeper the lining will have to be designed as sloping retaining walls which under worst conditions will be subject to pressure due to saturated backfill and differential water head across it. Arrangements should be made so that no water gets behind the linings from any external sources; and

iv. The backfill soil must be thoroughly compacted at the optimum moisture content, and if the canal is allowed to run for sometimes before the lining is laid it will further settle the soil specially where a canal runs in embankment;

Construction Joints and Expansion / Contraction Joints:

Construction of long canals and other structures, construction joints are provided in regular interval to prevent damages in the structures. There are mainly two types of construction joints.

Construction Joints

They are those which occur at points where, work having been stopped for any period, concrete already placed has started to harden or has hardened thus necessitating some form of jointing before any fresh concrete is placed. The concrete on either side of the joint, both old and new, should be quite dense. The position of construction joints should be such that the strength of the member is not affected.

Contraction and Expansion Joints

Contraction and expansion joints are necessary due to changes in volume of concrete caused by shrinkage due to:

- hydration of cement during settling;
- temperature changes, and change in moisture contains.

The reduction of water due to the gradual drying of the concrete results in shrinkage or contraction of the concrete. In the initial stages large stresses are built up in the concrete and these stresses must be relieved by incorporation of joints. Subsequently, when concrete is hardened, expansion and contraction occurs due to the seasonal variations of temperatures and moisture content. Unless a reinforced concrete structure is free to expand and contract, stresses are set up in the structure. The intensity of such stress depends on the range of temperature and variation in the moisture. Joints are therefore provided in the hydraulic structures.

Contraction joints are essentially breaks in the structural continuity of the concrete, and are intended to open when the concrete contracts during setting or when the temperature falls below the temperature of laying. Expansion joints permit the concrete to expand and contract. As concrete is very much weaker in tension than the compression, contraction joints normally have to be spaced at closer intervals than expansion joints.

Contraction is more dangerous than expansion because shrinkage leads to tension cracks through when water seepages. The magnitude of expansion is ½ to 1/3rd than of shrinkage or contraction.

Spacing between the construction joints depends on the type of the structure, type of concrete and the temperature variation in the area. In general, the expansion joints for the reinforced concrete dry walls may be 20 to 25 m interval. In un-reinforced walls, joints at intervals of about 4.5 m may be necessary for the walls exposed to weather and 9 meter for the wall protected from weather.

The construction joints as well as temperature joints (Expansion and contraction) will have to be filled. The filling materials may be bitumen impregnated fiber board, soft wood free from knots cork; self sealing synthetic rubbers packing for expansion joints etc are now available for the purpose.

Basic Formulae Used for the Canal Design

Hydraulic Calculation of Free flowing Canal

Open channel hydraulic design is of particular importance to hydropower design because of its interrelationship to most hydropower projects. The hydraulic principles of open channel flow are based on steady state uniform flow conditions. Though these conditions are rarely achieved in the field, generally the variation in channel properties is sufficiently small that the use of uniform flow theory will yield sufficiently accurate results.

Flow Classifications

(1) *Steady vs. Unsteady Flow:* The flow in an open channel can be classified as steady or unsteady. The flow is said to be steady if the depth of flow at a section, for a given discharge, is constant with respect to time. The flow is considered unsteady if the depth of flow varies with respect to time.

(2) *Uniform Flow:* Steady flow can further be classified as uniform or non-uniform. The flow is said to be uniform if the depth of flow and quantity of water are constant at every section of the channel under consideration. Uniform flow can be maintained only when the shape, size, roughness and slope of the channel are constant. Under uniform flow conditions, the depth and mean velocity of flow is said to be normal. Under these conditions the water surface and flow lines will be parallel to the channel bed and a hydrostatic pressure condition will exist, the pressure at a given section will vary linearly with depth.

(3) *Non-uniform Flow:* There are two types of steady state non-uniform flow:

- Gradually varied flow.

  Gradually varied flow is described as a steady state flow condition where the depth of water varies gradually over the length of the channel. Under this condition, the streamlines of flow are practically parallel and therefore, the assumption of hydrostatic pressure distribution is valid and uniform flow principles can be used to analyze the flow conditions.

- Rapidly varied flow.

  With the rapidly varied flow condition, there is a pronounced curvature of the flow streamlines and the assumption of hydrostatic pressure distribution is no longer valid, even for the continuous flow profile. A number of empirical procedures have been developed to address the various phenomena of rapidly varied flow. For additional discussion on the topic of rapidly varied flow, refer to "Open- Channel Hydraulics" by Chow.

Open Channel Flow Equations

The equations of open channel flow are based on uniform flow conditions. Some of these equations have been derived using basic conservation laws (e.g. conservation of energy) whereas others have been derived using an empirical approach.

(1) *Continuity Equation:* One of the fundamental concepts which must be satisfied in all flow problems is the continuity of flow. The continuity equation states that the mass of fluid per unit time passing every section in a stream of fluid is constant. The continuity equation may be expressed as follows:

$$Q = V_1 A_1 = V_2 A_2 = V_n A_n \qquad \text{------------------------------------} \qquad (3.21)$$

Where: Q is the discharge, A is the cross- sectional flow area, and V is the mean flow velocity. This equation is not valid for spatially varied flow, i.e., where flow is entering or leaving along the length of channel under consideration.

(2) Bernoulli Equation: Water flowing in an open channel possesses two kinds of energy: (1) potential energy and (2) kinetic energy. Potential energy is due to the position of the water surface above some datum. Kinetic energy is due to the energy of the moving water. The total energy at a given section as expressed by the Bernoulli equation is equal to:

$$H = z + h + \frac{V^2}{2g} \quad \text{------------------------------------} \quad (3.22)$$

Where,

H = Total head, in meter of water
z = Height above some datum, in meter
h = Depth of flow in meter
$\frac{V^2}{2g}$ = Velocity head in meter
g = Acceleration due to gravity = 9.81 m/s^2

(3) Energy Equation: The basic principle used most often in hydraulic analysis is conservation of energy or the energy equation. For uniform flow conditions, the energy equation states that the energy at one section of a channel is equal to the energy at any downstream section plus the intervening energy losses. The energy equation, expressed in terms of the Bernoulli equation, is:

$$z_1 + h_1 + \frac{V_1^2}{2g} = z_2 + h_2 + \frac{V_2^2}{2g} + h_L \quad \text{-------------------------------} \quad (3.23)$$

where,

h_L = Intervening head losses, in meters

(4) Manning's Equation: Several equations have been empirically derived for computing the average flow velocity within an open channel. One such equation is the Manning's Equation. Assuming uniform and turbulent flow conditions, the mean flow velocity in an open channel can be computed as:

$$V = \frac{1}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}} \quad \text{------------------------------------} \quad (3.24)$$

Where,

V = Mean velocity, in meters per second
n = Manning's coefficient of roughness
S = Channel slope, in meters per meter
R = Hydraulic radius, in meters = A/W_P
A = Cross sectional flow area, in square meters
W_p = Wetted perimeter, in meters

Commonly accepted values for Manning's roughness coefficient, n, based on materials and workmanship required in the Standard Specifications, are provided in Table 3.1. Direct solutions for Manning's equation for many channels of trapezoidal, rectangular, and circular cross sections can be found in FHWA's Hydraulic Design Series No. 3, "Design Charts for Open Channel Flow".

Among all the above mentioned equations, the most commonly used and widely accepted approach for the discharge calculation in open channel is Manning's equation.

Surge in the Canal: Canal surge may be defined as the hydraulic transients wherein the flows and pressures change in the conduit on account of the load variations in the hydel station units. The direct result of the surges is the sudden variations of water level in the channel. Canal surges are usually calculated using the following formula:

$$h_{\max} = \sqrt{\frac{V^4}{4g^2} + \frac{V^2 Z}{g}} \quad \text{for sudden closer} \quad \text{--------------------------} \quad (3.25)$$

$$h_{\max} = \frac{V^2}{4g} + V * \sqrt{\frac{Z}{g}} \quad \text{for gradual closer} \quad \text{---------------------------} \quad (3.26)$$

Where,

$h_{\max}$ = maximum surge

V = mean velocity of flow

Z = effective depth

Froude Number: The Froude number is a useful parameter which uniquely describes open flow. The Froude (F_r) number is a dimensionless value:

$$F_r = \frac{V}{\sqrt{g * d_m}} \quad \text{------------------------------------} \quad (3.27)$$

Where,

F_r = Froude Number

d_m = *A/T* = Hydraulic depth, in meters

A = Cross sectional area, in square meters

T = Top width of water surface, in meters

The state of flow, Super-critical, Sub-critical and Critical is determined by the Froude number which is given by:

if, Fr =1, Critical Flow
Fr >1, Super-critical flow
Fr <1, Sub-critical Flow

Hydraulic jump: When a rapid change in the depth of flow is from a low stage to a high stage, the result is usually an abrupt rise of water surface. This local phenomenon is known as a hydraulic jump. It occurs frequently in a canal below a regulating sluice, at the foot of a spillway, or at the place where a steep channel slope suddenly turns flat.

If the jump is low, that is, if the change in depth is smaller the water will not rise obviously and abruptly but will pass from the low to high stage through a series of undulations gradually diminishing in size. Such a low jump is called undular jump.

When a jump is high, that is, when a change in depth is great, the jump is called a direct jump. The direct jump involves a relatively large amount of energy loss through dissipation in the turbulent body of water in the jump. Consequently, the energy content in the flow after the jump is appreciably less than that before the jump. It may be noted that the depth before the jump is always less than the depth after the jump. The depth before the jump is called initial depth (h_1) and that after the jump is called the sequent depth (h_2).

Hydraulic Jump can be calculated using the following formula:

$$\frac{h_1}{h_2} = \frac{1}{2}\left(\sqrt{1 + 8F_2^{\,2}} - 1\right) \quad \text{------------------------------------} \quad (3.28)$$

$$\frac{h_2}{h_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^{\,2}} - 1\right) \quad \text{------------------------------------} \quad (3.29)$$

Where, F_1 and F_2 are the corresponding Froude number for h_1 and h_2.

Critical Flow

A useful concept in hydraulic analysis is that of "specific energy". The specific energy at a given section is defined as the total energy, or total head, of the flowing water with respect to the channel bottom. For a channel of small slope;

$$E = h + \frac{V^2}{2g} \qquad \text{------------------------------------} \qquad (3.30)$$

where,

E = Specific energy, in meters

h = Depth of flow, in meters

$\frac{V^2}{2g}$ = Velocity head, in meters

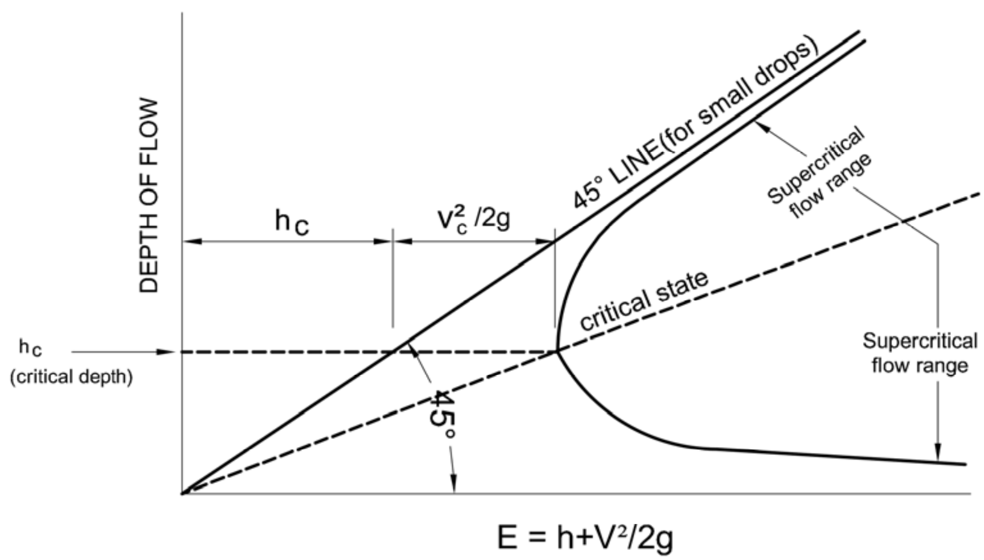


Fig. 3.6: Specific Energy Diagram

When the depth of flow is plotted against the specific energy, for a given discharge and channel section, the resulting plot is called a specific energy diagram (Fig. 3.6). The curve shows that for a given specific energy there are two possible depths, a high stage and a low stage. These flow depths are called alternate depths. Starting at the upper right of the curve with a large depth and small velocity, the specific energy decreases with a decrease in depth, reaching a minimum energy content at a depth of flow known as critical depth. A further decrease in flow depth results in a rapid increase in specific energy.

Flow at critical depth is called critical flow. The flow velocity at critical depth is called critical velocity. The channel slope which produces critical depth and critical velocity for a given discharge is the critical slope.

Uniform flow within approximately 10 percent of critical depth is unstable and should be avoided in design, if possible. The reason for this can be seen by referring to the specific energy diagram. As the flow approaches critical depth from either limb of the curve, a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of the specific energy curve. If the unstable flow region cannot be avoided in design, the least favorable type of flow should be assumed for the design. Critical depth is an important hydraulic parameter because it is always a hydraulic control. Hydraulic controls are points along the channel where the water level or depth of flow is limited to a predetermined level or can be computed directly from the quantity of flow. Flow must pass through critical depth in going from sub-critical flow to supercritical flow. Critical Velocity is given by:

$$V_c = \sqrt{g * h_{mc}} \qquad \text{------------------------------} \quad (3.31)$$

Where, Critical depth h_c is equal to the mean depth, h_{mc}, mean depth of the section corresponding to the critical depth.

Energy Dissipaters: Energy dissipaters are mostly used downstream of the weir, spillway or other hydraulic structures where water acquires very high velocity because of the conversion of the potential energy possessed by the water on the upstream side to kinetic energy as it flows down. If the water flowing with such a high velocity is discharged into the river or in any structures, it will scour the landing bed (river bed or other bed) in the water conveyance system of hydropower projects, energy dissipaters are used at the downstream of spillway constructed in the approach canal between the intake and desanding basin, spillway at forebay and some other outfall structures etc[5].

The design of the energy dissipater probably includes more options than any other phase of spillway design. The selection of the type and design details of the dissipater is largely dependent upon the pertinent characteristics of the site, the magnitude of energy to be dissipated, and to a lesser extent upon the duration and frequency of spillway use. Good judgment is imperative to assure that all requirements of the particular project are met. Regardless of the type of dissipater selected, any spillway energy dissipater must operate safely at high discharges for extended periods of time without having to be shut down for emergency repairs. The three most common types of energy dissipater used at hydropower projects are as follows:

a. The stilling basin which employs the hydraulic jump for energy dissipation.
b. The roller bucket which achieves energy dissipation in surface rollers over the bucket and ground rollers downstream of the bucket.
c. The flip bucket which deflects the flow downstream, thereby transferring the energy to a position where impact, turbulence, and resulting erosion will not jeopardize safety of the dam or appurtenant structures.

The design discharge for a given energy dissipater must be uniquely determined for each facility and should be dependent upon the damage consequences when the design discharge is exceeded. As a general rule, an energy dissipater should be designed to operate at maximum efficiency and essentially damage-free with discharges at least equal to the magnitude of the standard project discharge. The Kulekhani-I Dam energy dissipater is designed to contain the full spillway design flood (SDF) because the failure to do so would compromise the safety of the rock filled dam. The dissipater needs not be designed for the spillway design flood if operation with the spillway design flood does not create conditions endangering the dam or causing unacceptable economic damages.

Optimum energy dissipation will occur when the flow enters the dissipater uniformly. The hydraulic designer is responsible to ensure that project operating schedules are developed to maintain balanced flow operation of a gated, multiple-bay spillway at equal gate openings if the spillway is designed to carry large discharges. The designer must realize, however, that conditions may occur that require unbalanced operation, e.g., operator's error, or emergencies. Such conditions should be considered during evaluation of energy dissipation and stilling basin performance under conditions of non uniform flow distribution.

The flow chart for the design of canal is shown in Chart 1 below.

[5] Please note that the energy dissipators are more used in the Headworks design. For more detailed design of Energy dissipater: Refer to Spillway design or Small dam of USBR.

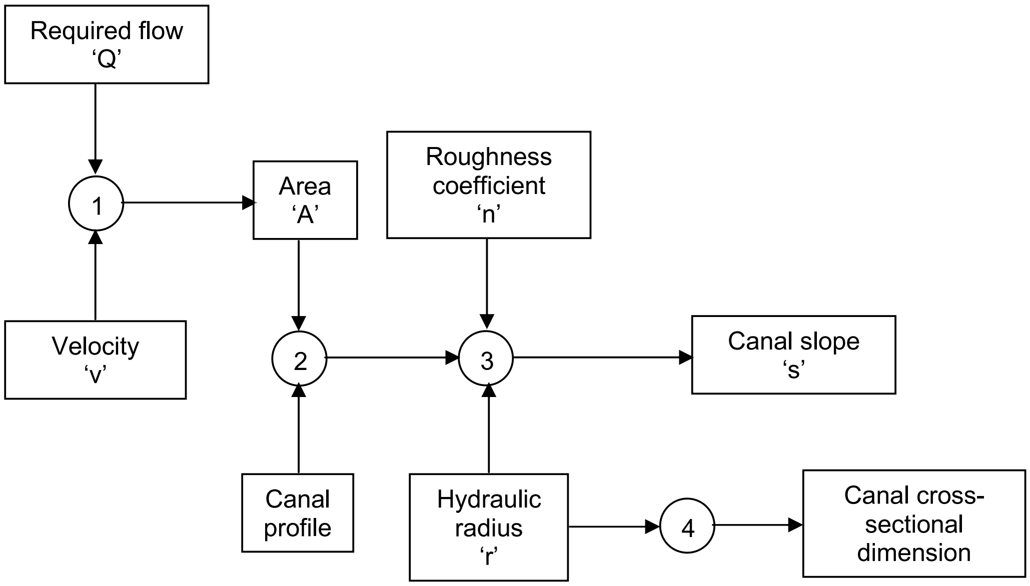


Chart 1: Flow Chart for Canal Design

3.1.1.4 Pipe

General

Very long pressurized pipes have been found generally used for supply of domestic and industrial waters. However, for smaller hydropower projects where unit construction cost of hydropower is of prime concern a long non or low pressure headrace pipes as water conveyance along geological fragile terrain ending at a stable forebay or surge tank have become, sometimes, more economical and practical. There is even an example of 3,487 m long penstock pipe connecting the headworks and hydropower plant on Piluwa Khola in the eastern Nepal for generating hydropower of 3 Mw capacity. A similar small hydropower plant (2.6 MW) in Small Sunkosi Hydropower Project, but designed with forebay has also a length of 2540 m after forebay. A 5 MW hydropower plant for Maya Khola, which is presently under going feasibility study, will have 7,436 m long low pressure pipe before the flow is discharged to a forebay. These examples show that at least in smaller scale hydropower project to be constructed in the mountainous regions of Nepal, the larger prospect for use of pipes as headrace water conveyance system persists. The flow in the pipe could be both non-pressured and pressured.

Water Flow in Pressure Conduits and Pipes

Hydraulic engineering is based on the principles of fluid mechanics, although many empirical relationships are applied to achieve practical engineering solutions. Until now there does not exist, and probably never will, a general methodology for the mathematical analysis of the movement of fluids. Based on the large amount of accumulated experience, certainly there are particular solutions to specific problems.

The energy in the water flowing in a closed conduit of circular cross section, under a certain pressure, is given by Bernoulli's equation:

$$H_1 = \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + h_1 \qquad \text{------------------------------------} \qquad (3.32)$$

Where H_1 is the total energy, h_1 is the elevation head, P_1 the pressure, γ the specific weight of water, V_1 the velocity of the water and g the gravitational acceleration. The total energy at point 1 is then the algebraic sum of the potential energy h_1, the pressure energy P/γ, and the kinetic energy $V_1^2/2g$.

Water is an incompressible fluid and its flow in pressure conduit and pipes are greatly influenced by flow characteristics, i.e. laminar or turbulent or transitional flow. The flow characteristic of flowing water in pressure conduits and pipes are classified by determining the Reynolds number which is represented by the equation:

Reynolds number(**R**) = $\dfrac{DV\rho}{\mu}$ ------------------------------------ (3.33)

Where,

V = Velocity, m/s
D = Diameter of pipe or conduit, m
ρ = density of fluid, kg/m^3
μ = absolute viscosity, N.s/m^2

In a pipe or conduit, the flow characteristic of water is classified as follow:

If **R** < 2000, the flow is **laminar**
If **R** > 2000 and < 3500, the flow is in **critical zone**
If **R >** 3500 and < 50000, the flow is **transitional** and
If **R** > 50000, the flow is **turbulent**

In pressure pipes or pressure conduits, the flow will be mostly turbulent and hence the boundary layer, viscous sub-layer and velocity profile in turbulent flow for pipes will be discussed in brief. The Moody's diagram (Fig. 3.7) presented below will represent the flow classification of water in pipes of different roughness.

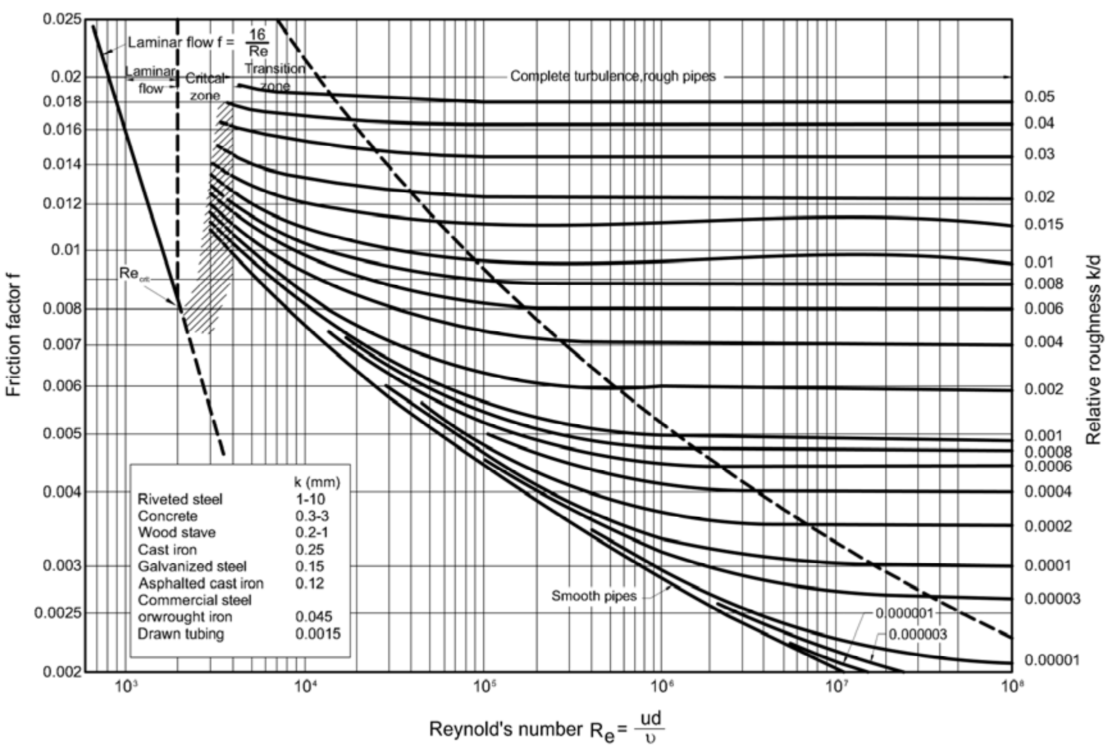


Figure 3.7: Moody Diagram

Viscous Sub-layer in Turbulent Flow for Pipe Flow

In a pipe, the initial boundary layer will be laminar at the entrance and fully developed turbulent flow will be found at about 50 pipe diameters from the pipe entrance for a smooth pipe with no special disturbance at entrance; otherwise the turbulent boundary layers from the two sides will meet within a shorter distance. It is fully developed turbulent flow that shall be considered in all this section that follows. The development of boundary layer is shown in Figure 3.8 below.

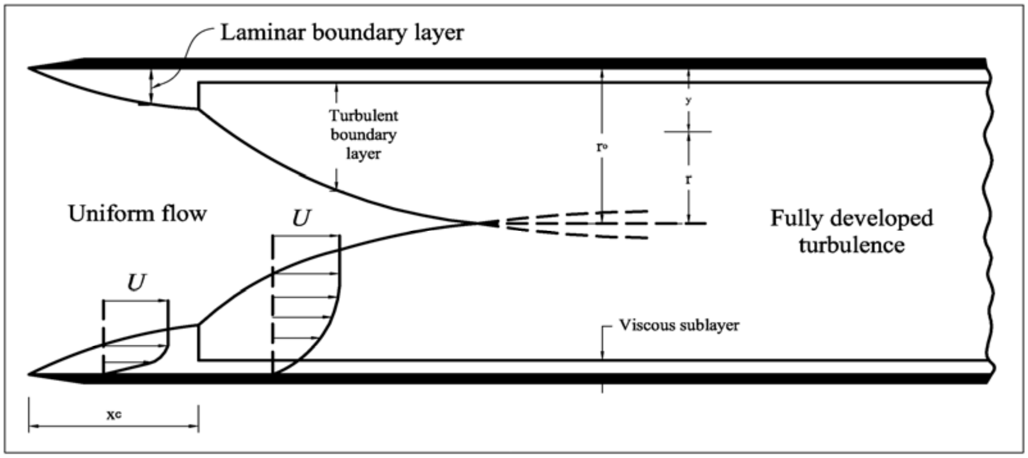


Figure 3.8: Development of Boundary Layer in a Pipe*

By plotting a velocity profile from the wall on the assumption that the flow is entirely turbulent, the velocity profile as shown below in Fig. 3.9 could be obtained.

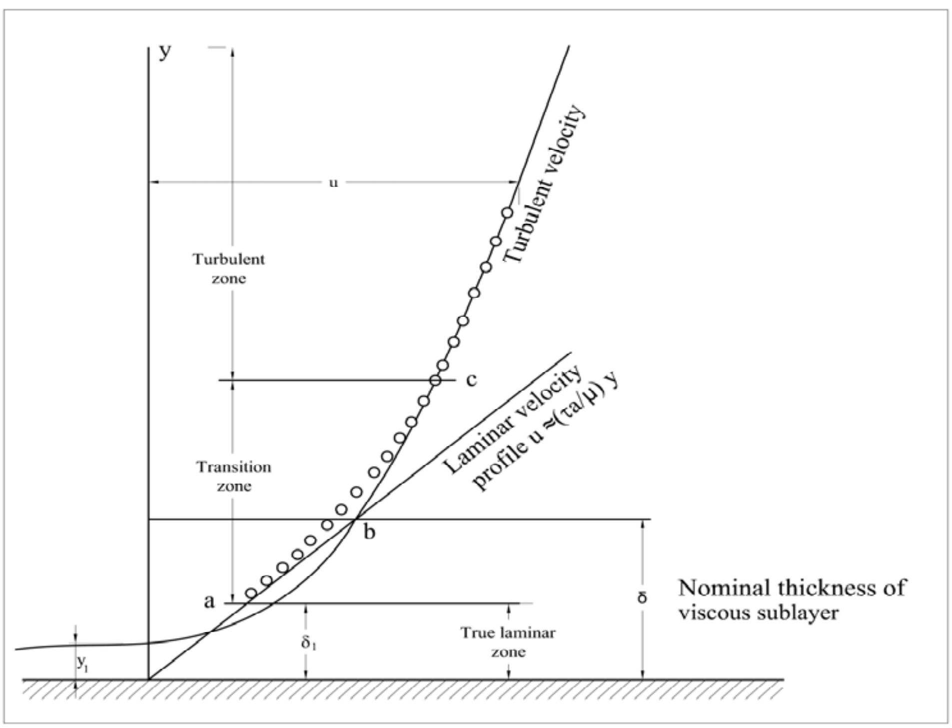


Figure 3.9: Velocity Profile near a Solid Wall*

* Source: Steady Incompressible Flow in Pressure Conduits, Fluid Mechanics with Engineering Applications, SI Metric Edition, ISBN 0-07-100405-X, by Robert L. Daugherty, Joseph B. Fransini, E. John Finnemore, SI metric Edition Adapted by Dr. K. Subramanya, McGraw-Hill Book Company.

The figures 3.8 and 3.9 represent the flow regime development (Turbulent and Laminar) and their significance in pressure pipes. The meanings of the notations or symbols used in these figures are presented below:

e	= height of surface roughness, mm
R	= Reynolds number = $LV\rho/\mu$ = LV/ν
r	= any radius, m or mm
r^o	= radius of the pipes, m or mm
y	= total depth of water in pipe or channel, m or mm
y_0	= depth for uniform flow in pipe or channel (normal depth), m
δ_1	= thickness of viscous sub-layer in turbulent flow, mm
δ	= thickness of boundary layer, mm
μ	= absolute or dynamic viscosity, N.s/m^2
ν	= kinematic viscocity, = μ/ρ , m^2/s
ρ	= density, mass per unit volume, kg/m^3
τ	= shear stress, N/m^2
$U or U_0$	= uniform velocity of fluid, m/s
u	= local velocity of fluid, m/s
u'	= turbulent velocity fluctuation in the direction of flow, m/s
V	= mean velocity of fluid, m/s
x_c	= distance measured from initial point, m

Thickness of viscous sub-layer

The thickness of viscous sub-layer is given by

$$\delta_1 = \frac{32.8\nu}{V\sqrt{f}} \qquad \text{------------------------------------} \qquad (3.34)$$

Where,
ν **=** kinematic viscosity, m^2/s
δ_1 = thickness of viscous sub-layer, m
V = Velocity, m/s
f = Friction factor

From above relationship, it is seen that the higher the velocity or lower the kinetic viscosity, the thinner the viscous sub-layer. Thus for a constant pipe diameter, the thickness of the viscous sub-layer decreases as the Reynolds number increases.

There is no such thing in reality as a mathematically smooth surface. But if irregularities on any actual surface are such that the effects of the projections do not pierce through the viscous sub-layer, the surface is hydraulically smooth from the fluid-mechanics viewpoint. If the effects of projections extend beyond the sub-layer, the laminar layer is broken up and the surface is no longer hydraulically smooth. To be more specific, if δ_1 > 6e, the pipe will behave as hydraulically smooth, while if δ_1 < 0.3e, the pipe will behave as wholly rough. In between these values, i.e., with 6e > δ_1 > 0.3e, the pipe will behave in a transitional mode; that is neither hydraulically smooth nor wholly rough. In as much as the thickness of the viscous sub-layer in a given pipe will decrease with an increase in Reynolds number, it is seen that the same pipe may be hydraulically smooth at low Reynolds numbers and rough at high Reynolds numbers. Thus, even a relatively smooth pipe may behave as a rough pipe if the Reynolds number is high enough. It is also apparent that, with increasing Reynolds number, there is a gradual transition from smooth to rough pipe flow. These concepts are depicted schematically in figure below, where e is the equivalent height of roughness projection.

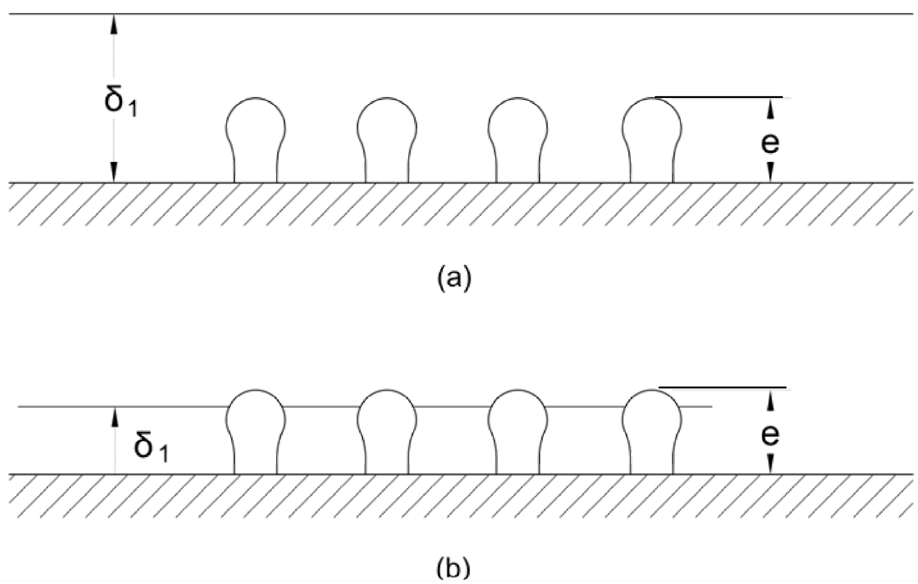


Figure 3.10: Turbulent flow near boundary (a) Relatively low R, if δ_1 > 6e, pipe behaves as smooth pipe. (b) Relatively high R, if δ_1 < 0.3e, pipe behaves as wholly rough pipe (Source: Steady Incompressible Flow in Pressure Conduits, Fluid Mechanics with Engineering Applications, SI Metric Edition, ISBN 0-07-100405-X, by Robert L. Daugherty, Joseph B. Fransini, E. John Finnemore, SI metric Edition Adapted by Dr. K. Subramanya, McGraw-Hill Book Company)

Velocity profile in Turbulent Flow

Prandtl reasoned that turbulent flow in a pipe is strongly influenced by the flow phenomena near the wall. He assumed that the mixing length near the wall was proportional to the distance from the wall.

The ratio of the mean to the maximum velocity may be obtained by the following relation:

$$\frac{V}{u_{\max}} \approx \frac{1}{1+1.33\sqrt{f}} \qquad (3.35)$$

Where,
V = Mean velocity = Q/A, m/s
$u_{\max}$ = Maximum velocity, m/s
f = friction factor = $\frac{h_l D(2g)}{LV^2}$ (Darcy-Weisbach equation)
h_1 =head loss, m
D = pipe Diameter, m
g = acceleration due to gravity, m/s^2
L = length of the pipe, m

For plotting the velocity profile for any mean velocity and any value of f in turbulent flow in pipe, the following equation will be applicable.

$$u = \left(1 + 1.33\sqrt{f}\right)V - 2.04\sqrt{f}\,V\log\left(\frac{r_0}{r_0 - r}\right) \text{ ---------------------} \quad (3.36)$$

where,

r_o = radius of the pipe and

r = arbitrary radius < r_0, (refer Figure 3.11)

The plotted velocity profiles for equal flow rates are presented in Figure 3.11 below:

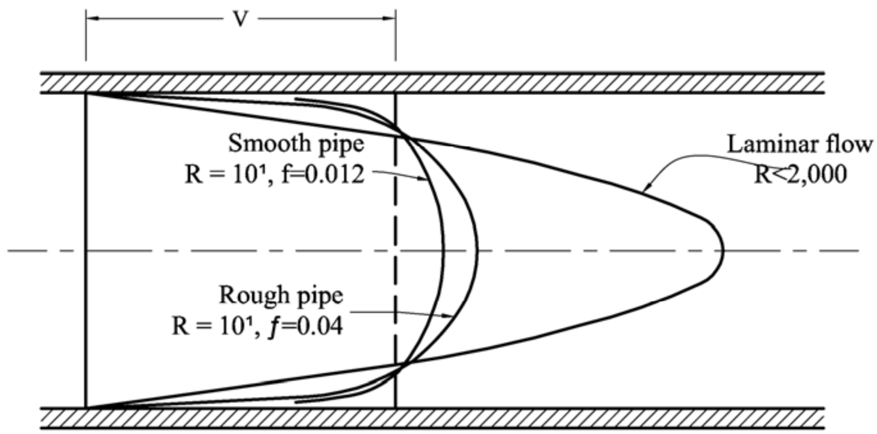


Figure 3.11: Velocity Profiles for Equal Flow Rates (Source: Steady Incompressible Flow in Pressure Conduits, Fluid Mechanics with Engineering Applications, SI Metric Edition, ISBN 0-07-100405-X, by Robert L. Daugherty, Joseph B. Fransini, E. John Finnemore, SI metric Edition Adapted by Dr. K. Subramanya, McGraw-Hill Book Company)

Pipe Roughness

Several experimenters have worked with pipes with artificial roughness produced by various means so that the roughness could be measured and described by geometrical factors, and it has been proved that the friction is dependent not only upon size and shape of the projections, but also upon their distribution or spacing.

The dimensional analysis of pipe flow showed that for a smooth walled pipe the friction factor '*f*' is a function of Reynolds number. A general approach, including absolute roughness 'e' as a parameter, reveals that '*f*' is a function of R and e/D. The term e/D is known as the relative roughness. From the experiment, it was found that in case of artificial roughness, the roughness is uniform, whereas in commercial pipes it is irregular both in size and in distribution. However, the roughness of commercial pipe may be described by e, which means that the pipe has the same value of '*f*' at high Reynolds number that would be obtained if a smooth pipe were coated with sand grains of uniform size e. The roughness factors for pipes are presented in Figure 3.12 below:

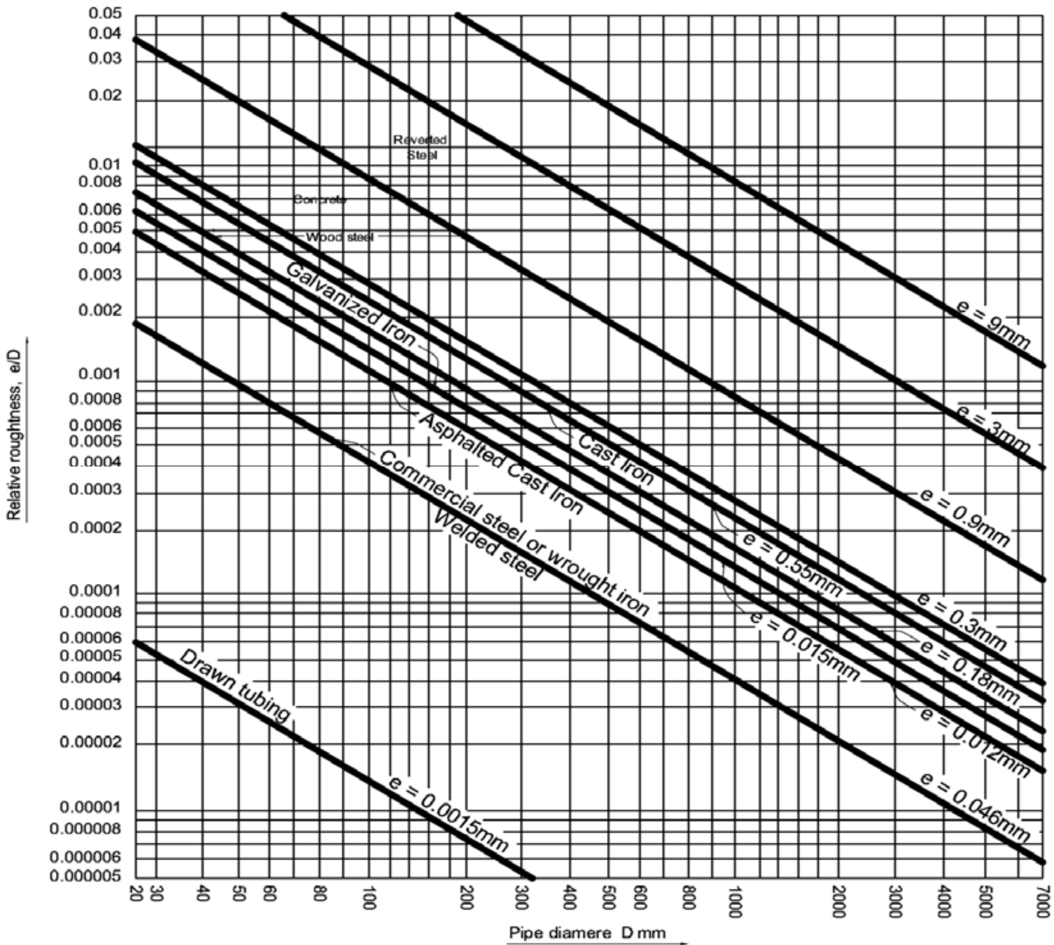


Figure 3.12: Roughness Factors (e* in mm) for Commercial Pipes (Source: Steady Incompressible Flow in Pressure Conduits, Fluid Mechanics with Engineering Applications, SI Metric Edition, ISBN 0-07-100405-X, by Robert L. Daugherty, Joseph B. Fransini, E. John Finnemore, SI metric Edition Adapted by Dr. K. Subramanya, McGraw-Hill Book Company).

Smooth Pipe flow

Von Karman equation for "smooth pipe" flow is given by the following relationship:

$$\frac{1}{\sqrt{f}} = 2logR\sqrt{f} - 0.8 \text{ (For } \delta_1 \text{ >6e) --------------------------------- (3.37)}$$

This equation applies to any pipe as long as δ_1 > 6e**;** when this condition prevails, the flow is known as smooth flow. The equation has been found to be reliable for smooth pipes for all values of **R** over 4000. Drawn tubing, brass, lead, glass, bituminous lining, centrifugally spun cement falls under the category of smooth pipes.

Rough Pipe flow

At high Reynolds numbers δ_1 becomes smaller. If δ_1 < 0.3e, it has been found that the pipe behaves as a wholly rough pipe; i.e. its friction factor is independent of the Reynolds number. For such case Von Karman found that the friction factor could be expressed as:

* This roughness factor "e" is also denoted as "k" in some books.

$$\frac{1}{\sqrt{f}} = 2\log\frac{D}{e} + 1.14 \quad \text{(For } \delta_1 < 0.3e\text{)} \text{ --------------------------- } \quad (3.38)$$

The value of f may be calculated with the help of Figure 3.13 and Table 3.8

Table 3.8: Values of absolute roughness e for new pipes

Pipe Material	Roughness e in millimeter (K)
Polyethylene	0.003
Welded steel pipe	0.6
Seamless commercial steel, light rust	0.25
Galvanized seamless commercial steel	0.15
Seamless commercial steel, new	0.025
Wood stave	0.6
Concrete(with steel form and smooth joint)	0.18
Asbestos cement	0.025
Fiberglass with epoxy	0.003

Transitional flow

In the gap where 6e > δ_1 > 0.3e neither smooth flow nor wholly rough flow applies. For such case Colebrook formula will be applicable and is given by

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{e/D}{3.7} + \frac{2.51}{R\sqrt{f}}\right) \quad \text{(For } 6e > \delta_1 > 0.3e\text{)} \text{ ----------- } \quad (3.39)$$

Illustrative Example: Water at 20^0C flows in a 0.5 meter welded steel pipe. If the energy gradient is 0.006, determine the flow rate (Q). Find also the nominal thickness (δ_1) of the viscous sub-layer.

Solution:

From Darcy-Weisbach Equation, $\left(h_f = \frac{fLV^2}{2gD}\right)$ we have

$$\frac{h_f}{L} = 0.006 = f\frac{V^2}{2gD} \text{ ------------------------------------ } \quad (3.40)$$

from which $V = \frac{0.243}{f^{0.5}}$

Try $f = 0.030$, then $V = 1.4$ m/s

and $R = \frac{DV}{\nu}$ = $7x10^5$

Using Moody diagram and Roughness factors chart,

For $R = 7x10^5$ and $\frac{e}{D} = \frac{.046}{500} = 0.00009$, $f = 0.0136$

For this case, $V = \frac{0.243}{f^{0.5}} = 2.08$ m/s and $R = 10^6$

For $R = 10^6$, the Moody Chart indicates $f = 0.0131$

For the next trial, let $f = 0.0131$. This gives $V = 2.12$ m/s and R is still $\approx 10^6$; hence $V = 2.12$ m/s is the answer.

$$Q = AV = \pi \frac{(0.5)^2}{4}(2.12) = 0.416 \text{ m}^3\text{/s}$$

Also,

$$\delta_1 = \frac{32.8\nu}{V\sqrt{f}} = \frac{32.8(10^{-6})}{2.12\sqrt{0.0131}}$$

Therefore,

$$\delta_1 = 135x10^{-6} \text{ m} = 0.135 \text{ mm}$$

Design Consideration

For flow calculation of non-pressure pipe, the design procedure applied for the open canal described in section 3.1 above using the formula for the circular section of canal could be applied, while for the pressure pipes, the design procedure could be as follows:

- **Alignment**

The alignment of the pipe should be straight and short to the extent possible and must be coordinated with other features including desanding basin, forebay / surge tank.

- **Suction Head requirement**

Pressure pipe must maintain suction head at its entrance to avoid air entering into the pipe which creates an unnecessary complication for the smooth flow. The minimum suction head required for the pipe flow is:

$$S = \mu * d * \frac{V}{\sqrt{g * d}} \qquad \text{--} \qquad (3.41)$$

Where,

S	-	Suction head, m
d	-	Pipe diameter, m
V	-	Velocity in the pipe, m/s
μ	-	Co-efficient whose value is 1.8 for Symmetric and 2.2 for Lateral
g	-	Acceleration due to gravity, m/s^2

Design Velocities

<u>Maximum Velocities</u>

The choice of velocity results from a compromise between a high velocity to reduce the diameter and cost of the pipe and the higher cost of water hammer protection associated with higher velocities and headlosses associated with it. Recommended maximum velocities for flow in low pressure pipelines are in the range of 1.3 to 1.5 m/s (ASAE, 1989). This applies to the mortar-jointed non-reinforced concrete pipe, low pressure asbestos cement and thin walled UPVC materials. But, depending upon the material of pipe maximum velocities are determined. The maximum velocity for commercial pipes should be limited to 3 m/s.

<u>Minimum Velocities</u>

Recommended minimum velocities are based on the need to ensure that any sediment or debris entering the pipe system is flushed during normal operation. Standard texts indicate scouring velocities for non-cohesive materials in the range of 0.3 m/s for silts and 0.5 m/s for fine sands, which are likely to be the most troublesome materials entering pipelines. A velocity of 0.5 m/s is acceptable for pipes of 350 mm diameter or greater, but over velocities are permitted for smaller pipe sizes.

- **Pipe Diameter Optimization**

After finalizing the alignment the economical diameter of the pipe has to be determined. Following factors should be considered while working out the economical diameter:

- Velocity requirement
- Head loss
- Interest on capital cost
- Annual maintenance charge

It is based on the increment of pipe cost with respect to the pipe diameter and the value of energy lost which is a function of the pipe diameter. A larger diameter for a given discharge leads to smaller head losses and hence greater will be the net head available for the energy generation. Thus the power and energy production will be increased. On the other hand a greater size pipe means less velocity and greater the capital investment. Therefore, a size that will give the least capital cost over the lifetime of the plant is considered to be the optimum diameter.

- **Economic Diameter and Shell Thickness**

The costs associated as mentioned above vary with time, especially the value of power and energy, so relatively complex computer calculations are necessary. This may be done to within ± 20% of final value by empirical equation developed by Fahlbusch (1987) through analysis of 394 steel-lined and concrete lined conduits for conventional and pumped-storage hydropower plants. For steel-lined conduits, he found the following relationship:

$D = 1.12\left(\frac{Q^{0.45}}{H_n^{\ 0.12}}\right)$, for D and H_n in meters and Q in m^3/s. -------------------------- (3.42)

And for a concrete-lined conduit
$D = 0.62Q^{0.48}$ for D in meters and Q in m^3/s.

Since the formulae for economic diameter and shell thickness of penstock pipes are also applicable for pipes and closed conduits, the details presented in Section 3.1.1.8 and 3.2.2.7 could also be referred.

- **Head loss in pipe flow**

Total head loss in the pipe is categorized in two major parts;

- Friction loss in length
- Local loss (bend loss, entrance loss, transition loss, etc)

i. Friction loss in length

Formula for the friction loss in length for the pipe depends on the nature of the flow in the pipe.

a. Laminar Flow:

Friction loss in length for the Laminar flow is given by Poiseuille's equation:

$$h_f = \frac{32\mu VL}{\rho g D^2} = \frac{32 v VL}{g D^2} \qquad \text{-------------------------------------} \qquad (3.43)$$

Where μ is coefficient of viscosity, ρ is density and D is internal diameter of pipe.

Friction loss in length for the laminar flow is independent of the pipe wall roughness and the friction gradient h_f / L is directly proportional to the mean pipe velocity. But the Laminar flow is unaffected by the nature of boundary surface.

b. Turbulent Flow

Friction loss in length in turbulent flow is calculated using number of formulae. Among them the Darcy Weisbach formula appears to be in common usage by all laboratories and engineering firms. Darcy Weisbach has introduced a pipe friction loss formula as below:

$$h_f = \frac{fLV^2}{2gD} \quad \text{------------------------------------} \quad (3.44)$$

f is the dimension less coefficient called friction factor, D is inside diameter of the pipe.

It is derived considering the shear stress of the pipe boundary. Process of obtaining the value of friction factor gradually developed with time. Most of the formulae for obtaining friction factor are based on the complex mathematical approach. Moody's diagram and value of k for different pipe materials are given in Figure 3.7 above in section 3.1.1.4.

ii. Empirical Formulae for the friction factor:

Prior to the publication of logarithmic formula the only design equations available were those of purely empirical exponential type. Simplicity is their chief merit, since they are particularly amenable to solution by means of nomograms and charts. They have been and still used extensively. Among the most used empirical formulae, Manning's formula for the friction factor as follows is most popular:

$$f = 124\frac{n^2}{D^{\frac{1}{3}}} \quad \text{------------------------------------} \quad (3.45)$$

The value of n here is the value of Manning's coefficient that is widely used.

It should be noted that Manning's formula is not dimensionless as a result large variations in the coefficient can occur for experiments on different size structures. Statements appear in literatures that the Manning's coefficient does not show the variation in the values for a particular instance that is exhibited by the Darcy formula. This is actually a deficiency.

Since the formulae are available to compute the value of friction factor / coefficient for any specific application and the values of roughness are also available for any number of surfaces (Ref. Table 3.9). It is not always necessary to depend on simplified single formula. The other empirical equations for pipe flow are given below:

Manning's Formula

$$\mathrm{V} = \frac{0.397}{\mathrm{n}}\mathrm{D}^{0.667}\mathrm{s}^{.5} \quad \text{-------------------------------} \quad (3.46)$$

and

$$h_f = 6.37n^2\frac{LV^2}{D^{1.33}} \quad \text{-------------------------------} \quad (3.47)$$

Where,

D = Inside diameter of pipe, m
n = Manning's coefficient
s = hydraulic gradient
V = mean velocity, m/s
h_f = head loss, m
L = length of pipe, m

Table 3.9: Manning's coefficient n for several commercial pipes

Kind of pipe	Manning coefficient n
Welded steel	0.012
Polyethylene (PE)	0.009
PVC	0.009
Asbestos cement	0.011
Ductile iron	0.015
Cast iron	0.014
Wood-stave (new)	0.012
Concrete (steel forms smooth finish)	0.014

The Hazen-William's formula

Probably the most popular formula in current use among waterworks engineers is the Hazen-Williams formula and is given by

$$V = 0.849 C_r^{0.63} s^{0.54} \quad \text{-------------------------------- (3.48)}$$

and

$$h_f = \frac{10.65 Q^{1.852} L}{C^{1.852} D^{4.87}} \quad \text{-------------------------------- (3.49)}$$

Where,

D = Inside diameter of pipe, m
s = hydraulic gradient
V = mean velocity, m/s
h_f = head loss, m
L = length of pipe, m
C = Hazen-William's coefficient

Tests have shown that the value of the Hazen-William's roughness coefficient C (Table 3.10) is dependent not only on the surface roughness of the pipe interior but also on the diameter of the pipe. Flow measurements indicate that for pipe with smooth interior linings in good condition, the average value of C = 140 + 0.17d, where d is the nominal pipe diameter. Please note that, Hazen-William's formula will be applicable only to the flow of water in pipes larger than 50 mm in diameter and velocities less than 3 m/s.

Table 3.10: Hazen-William's coefficients

Pipe type	**Hazen-William's coefficients, C**
Asbestos cement	140
Cast iron new Concrete, Cast on site in steel forms	130 140
Concrete, Cast on site in wood forms	120
Centrifugal cast	135
Steel, Brush tar and asphalt	150
Steel New uncoated	150
Steel Riveted	110
Wood-stave (new)	120
Plastic pipes	135 – 140

Material Properties of Pipe

The materials used in pressure pipes have the properties as shown in Table 3.11

Table 3.11: Material Properties of Pipe

Material	Young's modulus of elasticity $E (N/m^2) \times 10^9$	Coefficient of linear expansion α $(m/m^0C) \times 10^{-6}$	Ultimate Tensile Strength $(N/m^2) \times 10^6$	Manning's Coefficient, n
Welded steel	206	12	400	0.012
Polyethylene	0.55	140	5	0.009
Polyvinyl chloride	2.75	54	13	0.009
Asbestos cement	n.a.	8.1	n.a.	0.011
Cast iron	78.5	10	140	0.014
Ductile iron	16.7	11	340	0.015

n.a. denotes not available

Trashracks

- **Velocity Requirement in Trashrack**

A major design consideration for trashracks is the approach velocity. This value is the overall trashrack area divided by the maximum design flow. The approach velocity for each structure has to be determined on a case by case basis. An approach velocity of 0.3 to 0.6 m/s is desirable but not always practical. The slow approach velocity reduces the tendency to collect debris against the racks, minimizes the possibility of trashrack vibration. To limit the physical size of the structure and thus decrease the capital cost, larger approach velocities of up to 1.5 to 2 m/s can be used.

The factors which influence the approach velocity are:

- Economics
- Safety considerations
- Preservation of fish
- Location of the trashrack structure within the system
- Amount of expected debris
- Submerged conditions
- Age of structure (i.e., a new structure would probably have more debris than an old structure)
- Type of usage (i.e., a canal outlet versus a river intake structure)
- The amount of sediment that the water may be carrying
- Intake hydraulics and eddy currents

If a trashrack is to be raked, special consideration must be made. Different raking methods require different slopes for the trashrack. The slope of trashrack varies 5 to 30 degrees with vertical.

- **Trash Bar Spacing**

The spacing of trashrack bars should be as large as feasible and the thickness as low as practicable to minimize head losses. Trash bar spacing is selected to suit the equipment being protected. The usual practice is to provide as large a clear opening as possible and still protect the downstream equipment. For Francis turbine, the trash bar spacing may be estimated from the following equations.

$$S = FD_s/n \qquad \text{------------------------------------} \qquad (3.50)$$

$$F = 0.8 - N_s/762 \qquad \text{------------------------------------} \qquad (3.51)$$

Where,
S = the clear space between trashrack bars for Francis turbines, m
F = dimensionless number
D_s = the runner discharge diameter, m
N_s = Specific speed
N = the number of runner blades (use 19 if not known)

The trash spacing in case of Impulse turbines must be less than the clear opening of the nozzle. The headloss through trashracks depends upon the shape, size and spacing of bars and velocity of flow.

The Bureau of Reclamation's equation to estimate the headloss coefficient for trashracks is as follows:

$$k_t = 1.45 - 0.45\frac{a_n}{a_g} - \left(\frac{a_n}{a_g}\right)^2 \qquad \text{-------------------------------------} \qquad (3.52)$$

Where,

k_t = Trashrack loss coefficient

a_n = Net area through the rack bars, and

a_g = Gross area of the rack and supports

3.1.1.5 Tunnel

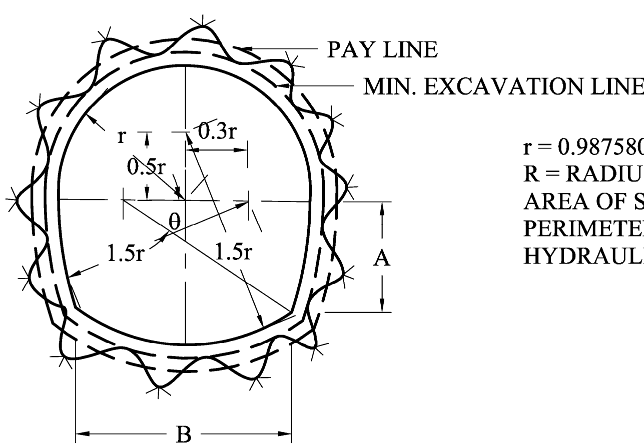
General

Conveyance tunnels for hydropower plants can be distinguished into two main groups. Those which discharge with full sections directly to the turbines are referred to as pressure tunnels. Owing to the considerable difference in elevation they are subjected to very large internal pressure. In contrast, tunnels constructed for the sole function of conveying water by normal gravity from one side of a mountain to the other, or from the point of diversion to the head of the steep incline, are termed discharge tunnels.

Between the two groups significant differences exit as far as loads, cross-section and shape are concerned.

Pressure tunnels are subjected to an internal water pressure which is frequently many times in excess of the external rock and groundwater pressure. The resultant tensile stresses can be resisted most economically by a circular cross-section. Pressure tunnels are therefore built with a circular or horseshoe-shaped cross-section, the latter being more readily adaptable to drilling methods. These tunnels cannot be built, unless the rock is completely immobile and solid, i.e. pressure tunnels, or pressure shafts must not be constructed in rocks interlaced with faults and cracks and tending to slip, or in those liable to tectonic movements.

Simple discharge tunnels, on the other hand, usually designed with a modified horse-shoe cross-section, the internal pressure executed by the water conveyed is negligibly small in comparison with the rock pressure acting on the tunnel. But the application of a reliable interior water seal is essential. A typical modified horse-shoe section of the tunnel is shown in Fig. 3.13.



r = 0.987580 R
R = RADIUS OF HYDRULICALLY EQUIVALENT CIRCLE

AREA OF SECTION	= 3.253 572 r ²
PERIMETER OF SECTION	= 0.426 334 r
HYDRAULIC RADIUS	= 0.506 287 r
A	= 0.780 776 r
B	= 1.561 553 r
θ	= 31°-22'-01"

Figure 3.13: A Modified Horse-shoe Section

Selection of Alignment

The position of the inlet portal, outlet portal, forebay, surge tank and surge chamber usually determines the location of the tunnel alignment despite the relation of orientation of the rock attitude with the tunnel axis. In general sense, the tunnel alignment connecting those locations will be shortest and the most economical condition. Hence in majority of situation the tunnel alignment should be selected accordingly. Under condition where the rock bedding / joints dipping at 45^0 to 90^0 is almost parallel to the tunnel axis, the shortest tunnel route might face very unfavorable geological condition resulting much instability problems and more support requirement. In such situation selection of the tunnel route oblique to the shortest tunnel axis will be preferable as far as possible.

The planning and investigation of hydraulic tunnels shall include the selection of its alignment and the geological competence along the alignment selected. Following points should be considered during the tunnel alignment selection:

- It should be shortest as far as possible-this would ensure minimum loss and shall be economically cheapest,
- It should be straight as far as possible- introduction of bends in the alignment shall involve losses at all such bends and the cost of tunneling would increase,
- It should be easily accessible- an easy access near the entrance and exit to the tunnel becomes essential for the construction facility,
- Careful selection of entry and the exit locations with minimum length and depth of approach cutting and no weathered, loose fractured layers slope towards portal.

However it is not always possible to follow a straight alignment because of the following parameters affecting the design of hydraulic tunnels:

- Topography: there may not be sufficient vertical and or lateral cover,
- Geological section along the alignment: it may show certain difficult strata through which the process of tunneling may be cumbersome and uneconomical,
- Ground and/or rock water loads along the tunnel alignment may be excessive thus increasing the overall cost of the tunnel,
- Rock mechanics properties: the in-situ stresses, joint pattern shear strength, unconfirmed compressive strength, shear modulus of deformation etc may not be favorable along a particular alignment,
- Creep or tectonic movement along the tunnel: the tunnel may be passing through active faults or lineaments which could cause collapse of the tunnel.

In addition to the above mentioned problems, certain other parameters may also affect the tunnel alignment because of the construction difficulties, they are:

- Presence of methane gas,
- Presence of jointed rock aquifers confined by impervious strata which might lead to heavy inflow of water,
- Squeezing rock conditions.

Investigation

Geological

The investigations for selecting the route or alignment of tunnels are done by means of selective drilling and by making drifts and test tunnels to obtain as much information as possible of the geological conditions prevailing at the site. The most important phase of preliminary work in tunneling is the careful exploration of geological conditions prevailing at the site.

The purpose of geological explorations is as below:

- The determination of the origin and actual conditions of rocks,
- The collection of hydrological data and information of underground gases and soil temperatures,
- The determination of physical, mechanical and strength properties of rock along the proposed line of a tunnel,
- The determination of geological features which may affect the magnitude of rock pressure to be anticipated along the proposed locations.

Exploration should be extended to determine:

- Top cover,
- Quality of subsurface rock,
- Surface drainage conditions,
- Position, type and volume of water and gases contained in subsurface rock,
- The physical properties and resistance to tunneling offered by the encountered rock.

The sequence of geological explorations of tunnel may be divided into following three groups:

a) Investigation of a general character prior to planning, which should include the bibliographical and statistical survey morphology, petrography, stratigraphy and hydrology of the environment. This should be complemented by a thorough field reconnaissance and by surface exploration.
b) Detailed geotechnical (subsurface) investigation parallel to planning but prior to construction, by which an improved information should be obtained for detailed design.
c) Geological investigation should be continued during construction, not only in the interests of checking design data but also for ascertaining whether driving method adopted is correct or needs to be modified.

For detail design of the tunnel, investigation should include performance of detailed geological mapping, drilling, seismic refraction survey or 2D resistivity survey, test aditing and in-situ rock strength testing.

Surface geological mapping: Geological mapping of the ground surface should be given prime importance as it can deliver significant amount of information at minimum cost than that to be derived from the other direct investigation method such as drilling and test aditing, and from the indirect method like seismic refraction survey or 2D resistivity survey. Examination will be oriented to acquire information useful for the rock mass classification.

The geological / geotechnical investigation should be conducted covering the ground surface between the tunnel alignment, and its portals and adit level. Its coverage is preferable extending 100 m beyond the tunnel alignment and 50 m below the adit level. The results should be primarily based on the observation of the rock exposures with supplementation of finding of the 2D resistivity survey, seismic refraction survey, drilling and test aditing works. Information obtained should be presented in the form of the map and section in scale 1:5000 or 1:10,000 or 1:20,000 as appropriate.

Drilling: The feasibility study report contains generally the information of drilling only for the tunnel portals. It lacks information for the alignment. Though the need for drilling along the tunnel alignment was given much consideration, actually drill holes were rarely placed in the tunnel alignment due to deep ground cover and high cost involvement. In fact they could provide good site specific geological information for the tunnel level. Such result might only be representative of short tunnel length and will not be much useful where complexity of geology is apparent. Good outcrop geology will be highly valuable in such situation. However, penetration of the drill holes might be considered at the critical areas along the tunnel alignment provided the terrain condition is favourable to achieve the depth of tunnel elevation. The position of saddles might be appropriate site for such purpose.

Test Adit: Excavation of test adit is considered to be valuable means of investigation because it provides opportunity for the (i) first hand look at the ground of potential tunnel elevation, (ii) space for in-situ rock strength testing, (iii) application of different excavation techniques and supports, and (iv) availability of pre-construction tunnel access. It should be driven to investigate the sub-surface rock condition considered to be representative of different ground present along the tunnel alignment. It may be preferably excavated not only through some of the poorer ground along the tunnel route but also at the locations through which easy access to a crucial section of the main tunnel line is planned.

Test adits might be converted into the access adits provided they are situated at a favourable and convenient location for mobilization and transportation of the project personnel, equipments, construction materials and spoil tip.

Each adit location is carefully selected to ensure as far as possible that a stable portal site and suitable rock quality is present taking into account of the data obtained from the detailed surface geological mapping and seismic survey / 2D resistivity survey. In this context, adit locations for the hydropower development in Nepal, have been found / identified at different spacing depending upon the length of the tunnel. For an example they were placed at 0.25 km to 2.3 km apart in 5,847 m long tunnel of Kulekhani-II hydropower project, at 0.13 to 3.2 km apart in 7807 m long tunnel of Khimti hydropower project, at 0.5 km to 1.7 km apart in 3.44 long tunnel of Bhotekoshi hydropower project and at 0.2km to 10.8 km apart in 28.7 km long tunnel of the Melamchi Drinking Water Project. In consideration of the above facts, the adit should be driven in an objective to provide efficient and economic construction environment for the tunneling works.

Seismic Refraction Survey: In consideration of stability assessment of the tunnels portals, application of the seismic refraction survey is particularly necessary to determine the characteristics of the overburden materials, and the depth and quality of the underlying rocks. Such investigation will be more significant when the rock exposures are rarely observed close to the tunnel portals. It has limited application along the tunnel route when required to establish the quality of the rocks at greater depth of tunnel elevation. This condition is generally encountered in the tunnels of Nepal. The saddles, fault zone and other specific sites representing the particular rock type could be suitable locations for the purpose of deriving the rock quality in terms of seismic velocity. Consequently comparison of the findings with similar examination done at the test adit and surroundings will be fruitful to assess the rock quality along the tunnel alignment.

Resistivity Survey: The 2D resistivity survey can be considered as the substitute sub-surface exploration method to determine the nature of the overlying soil materials, and the depth of the bedrock and its quality at the tunnel portals where deep soil cover and weathered rocks are commonly existent. Its application is not much convincing to reveal the geological characteristics of the tunnel level at greater depth.

Hydrological Survey: The hydrological survey is carried out simultaneously with the geological exploration, since water is a governing factor in tunnel loads as well as in construction possibilities and conditions. The appearance of water in drifts and tunnels depends primarily on the character and distribution of water conveying passages.

Gases and Rock Temperatures: Another important part of the preliminary exploration is the estimation and study of gas outbursts, gas infiltration and rock temperatures. Gas and rock temperatures are significant factors for the safety and health of workmen.

Geometric Design of Tunnel

After the finalization of tunnel alignment and geological investigation, tunnel geometry is selected. There is no hard and fast rule for the tunnel geometry selection. Therefore, the tunnel geometry entirely depends on the prevailing geological condition and judgment of the designer. No general recommendations can be made to fit in each and every individual case but a few important tunnel geometry widely used in the hydraulic design are as follows:

- Circular section
- D-shaped section
- Horse-shoe section
- Modified horse shoe section

Although there is no general recommendation regarding the shape of the tunnel, its cross-section depends on the following factors:

- Prevailing geological conditions along the tunnel alignment,
- Hydraulic requirement,
- Structural requirements, and
- Functional requirements.

The final choice of the section is made by carefully scrutinizing the above four factors. General requirements and design parameters for the few widely used shape are discussed below:

Circular Section

Circular section is most suitable from structural considerations. However, it is difficult for excavation, particularly where the cross sectional area is small. In a case where the tunnel is subjected to high internal pressure but does not have good quality rock or adequate rock cover around it, circular section is considered to be most suitable.

D-Shaped Section

D shaped sections is suitable for tunnels located in good quality rocks. The main advantages of this section over horse shoe section are the added width of the invert which gives more working floor space in the tunnel during driving and flatter invert which helps to eliminate the tendency of wet concrete to slump and draw away from the tunnel sides. The added invert width also permits the use of concurrent lining of the tunnel which may not be possible for circular and horse-shoe tunnels of the same dimensions.

Horse-shoe or Modified Horse-shoe Sections

These sections are a compromise between circular and D shaped sections. These sections are structurally strong to withstand external rock and water pressure. These sections are found to be most suitable, where a moderately good rock is available, advantages of a flatter invert are required for construction purposes and the tunnel has to resist internal pressure. These modified horse-shoe sections also afford easy change over the circular sections with minimum additional cost in reaches where rock quality is poor or rock cover is inadequate.

Economical Diameter Studies / Tunnel Optimization

Following factors should be considered while working out the economical diameter:

- Velocity requirement,
- Head loss in tunnel,
- Interest of capital cost of tunnel,
- Annual operation and maintenance charge,
- Whether lined or unlined, and
- Cost of gates and their hoisting equipment.

It is based on the increment of tunnel cost with respect to the tunnel diameter (sectional area) and the value of energy lost which is a function of the tunnel sectional area. A larger diameter for a given discharge leads to smaller head losses and hence greater will be the net head available to the turbine. Thus the power and energy production will be increased. On the other hand a greater size tunnel means less velocity and greater capital investment. Therefore, a size that will give the least capital cost over the lifetime of the plant is considered to be the optimum diameter / sectional area. A typical curve for the tunnel optimization is shown in Fig. 3.14 below:

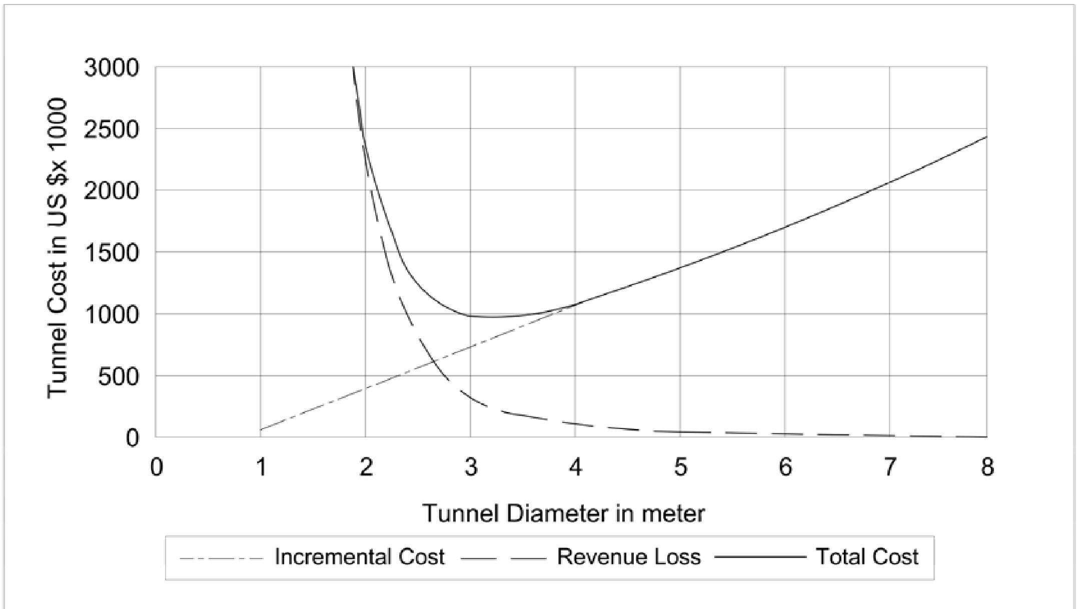


Fig.3.14: A Typical Curve for Tunnel Optimization

Hydraulic Design of the Tunnel

Basically, there are two flow conditions in the hydraulic tunnel design:

Free Flow Tunnel

Hydraulic design of the free flow tunnel is executed as the hydraulic design of canal and Manning's frictional factors are used to compute the head loss in length.

Pressure Flow Tunnel

Hydraulic design of the pressure flow is computed as the pipe flow and the head loss is computed using Darcy/Weisbach frictional factor. Discharge through a pressurized hydraulic tunnel is calculated using a continuity equation with control volume approach:

$$Q = V * A \qquad (3.53)$$

where,
Q = discharge in m^3/s
V = velocity in m/s
A = Area in m^2

and the optimization study with the permissible velocity in the given type of tunnel will determine the optimum size of tunnel for the given discharge. Optimization study comprises of annual charges composed of fixed cost, cost of operation and maintenance and value of energy losses. These can be expressed as a function of diameter and should be minimum for the economical diameter.

Head-loss in Tunnel

Head loss in Tunnel is also categorized into two major components like in pipes:

Friction loss in length

Friction loss (R_c) in length for tunnel is calculated either by using Manning's formula or Darcy-Weisbach formula as given below.

Manning's formula:

$$h_f = \frac{V^2 n^2 L}{R^{\frac{4}{3}}} \quad \text{------------------------------------} \quad (3.54)$$

Where,

h_f - Frictional loss

V - Velocity of water in the tunnel in m/s

L - Length of the tunnel in m

R - Hydraulic radius

n - Rougosity coefficient

For concrete lined tunnel the value of rougosity coefficient n varies from 0.012 to 0.018. For unlined tunnel the value of n depends upon the nature of rock and the quality of trimming. Recommended values of n for various rock surface conditions are given below:

Surface Characteristics	Minimum value of n	Maximum value of n
Very rough	0.04	0.06
Surface trimmed	0.025	0.035
Surfaced trimmed and invert concreted	0.02	0.03

Darcy Weisbach formula:

$$h_f = \frac{fLV^2}{2gD} \quad \text{------------------------------------} \quad (3.55)$$

Where,

h_f - Frictional loss

f - Friction coefficient

L - Length of the tunnel in m

D - Diameter of tunnel in m

V - Velocity of water in the tunnel in m/s

The friction coefficient *f* depends upon the Reynolds number and the relative roughness k_s/D where k_s is the equivalent sand grain roughness and its value depends upon the surface characteristics. For new concrete lined tunnels using forms the value of k_s varies from 0.015 mm to 0.18 mm. For welded steel lined tunnels, the value of k_s ranges from 0.05 to 0.1.

Because of fluctuation in the load demand, the turbines keep an accepting or rejecting water. This cause the flow in the water conductor system to be turbulent. For turbulent flow in the range of Reynolds number between 3000 to 10000 (normally expected in steel lined and concrete lined tunnel) the friction factor is calculated by using formula:

$$\frac{1}{f} = 2.0 \log_{10} \frac{1}{2E} + 1.74 \quad \text{------------------------------------} \quad (3.56)$$

where, *f* is the factor use in the Darcy formula and E is relative roughness given by:

$E = k_s/D$

For unlined tunnels, the value of f depends upon the variation in cross sectional area obtained in the field. The frictional loss factor may be estimated by measuring cross sectional areas at intervals and determining the value of *f* by the following formula:

$$f = 0.00257\delta \qquad \text{------------------------------------} \qquad (3.57)$$

where

$$\delta = \frac{A_{99} - A_1}{A_1} x100$$

A_{99} - area corresponding to 99 percent frequency

A_1 - area corresponding to 1 percent frequency

For tunnels of non-circular section, the diameter D in Darcy's formula shall be replaced by 4R where R is the hydraulic mean radius (A/P). The Darcy formula shall thus read as follows:

$$h_f = \frac{\lambda L V^2}{8gD} \qquad \text{------------------------------------} \qquad (3.58)$$

λ = friction loss coefficient

For tunnel flowing partly full, the head loss due to friction will be calculated by using Manning's formula given earlier.

Local and Transition Losses

The local losses include trash-rack loss, entrance loss, transition loss, bend/ junction losses, gate loss and exit loss.

Trash-rack loss: Tunnel openings are provided with trash racks at the intake to prevent the entry of floating debris into the tunnel. Where minimum loss value is desired, it is usual to assume 50 percent of the rack area as clogged. This would result in twice the velocity through the trash-rack. Since the loss varies directly as the square of the velocity, it is desirable to limit the velocity at the intake to about 1 m per second for the worst conditions. For maximum trash rack losses, the racks may not be considered clogged when computing the head loss, for the loss may be neglected altogether. The trash rack loss shall be computed by using the following formula:

$$h_t = k_t \frac{V^2}{2g} \qquad \text{------------------------------------} \qquad (3.59)$$

where,

h_t - trash-rack loss

k_t - loss coefficient for trash-rack calculated by the formula:

$$k_t = 1.45 - 0.45\frac{a_n}{a_t} - (\frac{a_n}{a_t})^2 \qquad \text{------------------------------------} \qquad (3.60)$$

where,

a_n - net area through the trash rack bars

a_t - gross area of the opening

V - Velocity

Entrance loss: To minimize the head losses and to avoid zones where cavitations pressures may develop, the entrance to a pressure tunnel should be streamlined to provide gradual and smooth changes in flow. For best efficiency the shape of the entrance should stimulate that of a jet discharge into air and should guide and support the jet with minimum interference until it is contracted to the tunnel dimensions.

For a circular tunnel the bell mouth entrance shape may be approximated by an elliptical entrance curve given by the equation:

$$\frac{x^2}{(0.5D)^2} + \frac{y^2}{(0.15D)^2} = 1 \qquad \text{------------------------------------} \qquad (3.61)$$

where, x and y are the coordinated axes and D is the tunnel diameter at the end of entrance transition. For headrace tunnels since a gate is essential at the entrance, the opening shall be either rectangular or square. For such an opening the elliptical curve for the entrance shall be approximated by the equation:

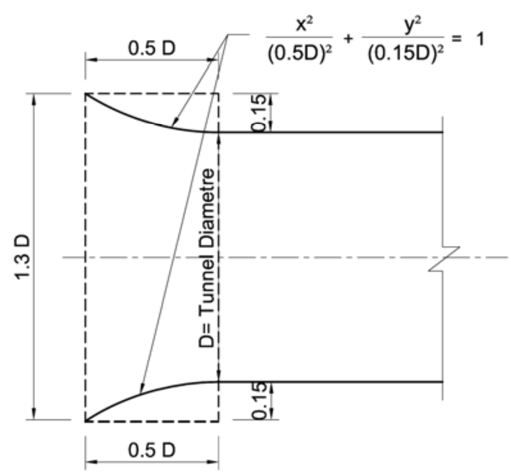
$$\frac{x^2}{(D)^2} + \frac{y^2}{(0.33D)^2} = 1 \qquad \text{------------------------------------} \qquad (3.62)$$

where D is the vertical height of the tunnel for defining the top and bottom curves and is also the horizontal width of the tunnel for defining the side curves.

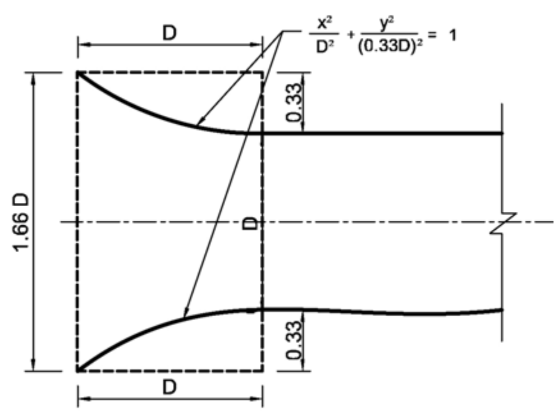
For rectangular entrance with the bottom place even with the upstream floor and with curved side piers at each side of the entrance openings, both the bottom and side contractions will take place at the top of the opening. For such a case, the top curve may be obtained from the equation:

$$\frac{x^2}{(D)^2} + \frac{y^2}{(0.67D)^2} = 1 \qquad \text{------------------------------------} \qquad (3.63)$$

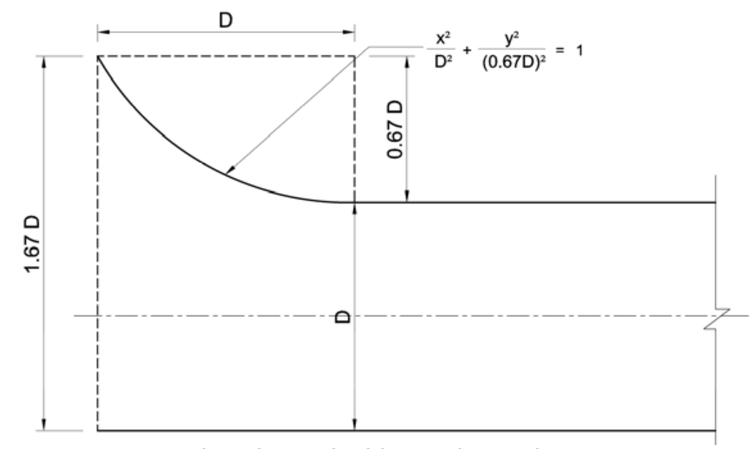
Where D is the vertical height of the tunnel downstream of the entrance.



CIRCULAR TUNNEL WITH BELL-MOUTH INTAKE



RECTANGULAR OR SQUARE OPENING



RECTANGULAR OR SQUARE OPENING WITH
BOTTOM AND SIDE CONTRACTIONS SUPPRESSED

Entrance loss shall be computed by using following formula:

$$h_e = k_e \frac{V^2}{2g} \quad \text{------------------------------------} \quad (3.64)$$

where,

k_e - loss coefficient for entrance

V - velocity of flow

The value of k_e for circular bell mouth entrance varies from 0.04 to 0.1 with an average value of 0.05 and that for square bell mouth entrance varies from 0.07 to 0.20 an average value of 0.16.

Transition loss: In hydraulic tunnel, transitions are often required at the intake, junctions with desanding chambers, gate galleries, surge shafts, etc, and at outlets. All these transitions cause head loss in tunnels. To minimize the head loss and to avoid cavitations tendencies along the tunnel surface, the transitions should be gradual. Transitions can either be for contraction or for expansion.

For contractions, the maximum convergent angle should not exceed than the relationship below:

$$\tan \alpha = \frac{1}{U} \quad \text{---} \quad (3.65)$$

Where,

α - Angle for the tunnel wall surface with respect to its centre line

U - An arbitrary parameter $= \dfrac{V}{\sqrt{gD}}$

V and D - average of the velocities and diameters at the beginning and end of the transitions.

Expansions should be more gradual than contractions because of the danger of cavitations where sharp changes in the side walls occur. Expansion angle should be based upon the following relationship:

$$\tan \alpha = \frac{1}{2U} \quad \text{------------------------------------} \quad (3.66)$$

It has been noticed that head loss increases rapidly in the case of expansion where the angle α exceeds 10^{o}. Hence, for all hydraulic tunnels and for pressure tunnels in particular, angle α must be limited to 10^{o}.

Where a circular tunnel flowing partly full discharges into a chute or channel, the transition from the circular section to the one with flat bottom may be made either within the tunnel itself or in the open channel downstream from the tunnel portal. The length of the transition for exit velocities of up to 6.0 m/s may be obtained by using the relationship:

$$L = \frac{2VD}{3} \quad \text{------------------------------------} \quad (3.67)$$

where,

L - Length of transition in m

V - Exit velocity in m/s

D - Tunnel diameter in m

For expanding transition, the head loss is given by the following formula:

$$h_t = k_e\left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right) \quad \text{------------------------------------} \quad (3.68)$$

where,

h_t - head loss in expanding transition in m

V_1 - average velocity in m/s at the beginning of transition

V_2 - average velocity m/s at the end of transition

k_e - loss coefficient for expansion given by

$$k_e = 3.50\left(\tan\frac{\alpha}{2}\right)^{1.22} \quad \text{------------------------------------} \quad (3.69)$$

α - Angle of the tunnel wall surface with respect to its centre line

For contractions, the head loss shall be computed using the following formula:

$$h_c = k_c\left(\frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g}\right) \quad \text{------------------------------------} \quad (3.70)$$

where,

h_c - head loss in contracting transition in m

V_1 - Velocity in the normal section m/s

V_2 - Velocity in the contracted section in m/s

k_c - loss coefficient for contraction

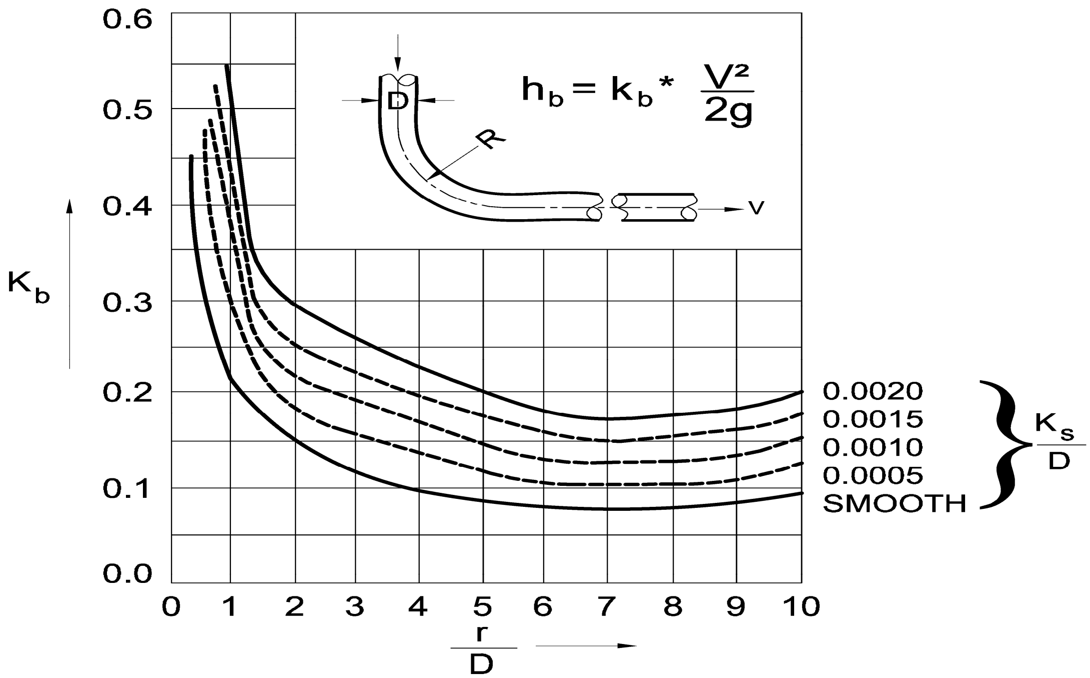
The value of k_c varies from 0.1 to 0.5. for gradual contraction where the flare angle does not exceed 10^0 the value of k_c shall be taken as 0.1.

Bend, Gate and Exit losses: Bend and junction in hydraulic tunnel is unavoidable owing to their functional and constructional requirements. These bends and junctions also cause loss of head which must also be computed.

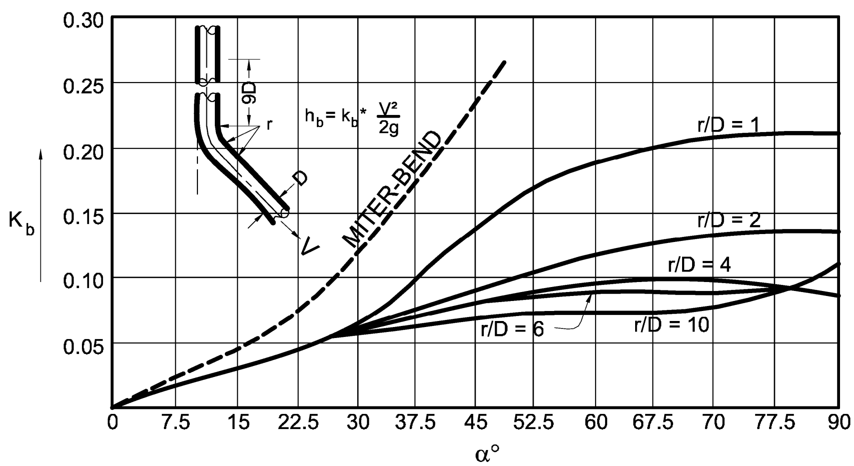
Bend loss depends upon the relative roughness k_s/D and r/D ration, where, k_s is the absolute roughness, D is the diameter of the tunnel and r is the radius of the bend. The head loss due to bend is given by:

$$h_b = k_b\left(\frac{V^2}{2g}\right) \quad \text{------------------------------------} \quad (3.71)$$

Knowing the value of k_s/D and r/D the value of K_b for 90^0 bends and that for bends with deflection angles other than 90^o may be obtained from below.



MINIMUM STRAIGHT LENGTH
Head loss Coefficient for 90^0 bend



MINIMUM STRAIGHT LENGTH
Head loss Coefficient for other than 90^0 bend

Gate loss: If the entrance to the tunnel is designed properly the velocity of flow would be approximately 1m/s. In such a case no gate loss needs to be considered. However, there will be head loss due to the gate groove and the gate shaft which is given by the formula:

$$h_g = k_g\left(\frac{V^2}{2g}\right) \qquad \text{------------------------------------} \qquad (3.72)$$

where,

h_g - gate head-loss in m

V - Velocity of the flow in m/s

k_g - loss coefficient for gate

The value of k_g can be assumed to be 0.10. For partly open gate, the value of k_g will depend upon the top contractions and it varies from 0.20 to 0.10.

Exit loss: Where no recovery of velocity head will occur, such as where the release from a pressure tunnel discharges freely or is submerged or supported on downstream floor, the velocity head loss coefficient k_{ex} shall be taken as unity. Head loss at the exit would be calculated by using the formula:

$$h_{ex} = k_{ex}\left(\frac{V^2}{2g}\right) \qquad \text{------------------------------------} \qquad (3.73)$$

where

h_{ex} - exit head-loss in m

V - Exit velocity in m/s

k_{ex} - loss coefficient for exit

Construction Adit

After the finalization of the tunnel alignment, it becomes necessary to decide the number of working faces for the tunnel. In each and every case of tunneling, at least two working faces- one on each end of the tunnel- are always available. Depending upon the length of tunnel and the urgency for its completion, the tunnel may be provided with one or more intermediates- these intermediates are called adit / adit tunnel. Each such adit would yield two more working faces through which tunneling could progress. For ease of mucking it is desirable to have the intermediate adits in a horizontal plane. However, depending upon the topography of the area, in certain cases it is possible to provide only vertical intermediate adit.

The horizontal adits which have no hydraulic function after the tunnel has been completed are plugged with mass concrete. The vertical adits often play a very important hydraulic function- that of acting as a small surge shaft which help in controlling surge in the tunnel and hence are never plugged.

Tunnel Portal

A tunnel portal is the very beginning of the tunnel excavation work. Whenever tunneling operation is to be started, a portal has to be constructed at the working face. Thus, it is obvious that an approach road has first to be constructed to reach the working face. It has been seen that generally rocks near the working face are highly weathered and the first few meters are nothing but loose overburden. This loose overburden is first of all removed and thus a working platform at the invert level of the proposed tunnel is made available. This outline of the tunnel face is then marked on the exposed rock face and an R.C.C. or steel framed portal is constructed around the periphery of the proposed tunnel. The main function of the portal is to provide a well defined access to the tunnel and to protect the tunnel face from loose overburden falling above the tunnel opening. The structural design of the portal is very simple. Generally the load coming over the beam of the portal is the self load of the beam plus a uniformly distributed live load calculated by assuming 45^0 dispersion above the beam. Design procedure of the tunnel portal is given in the reference books "The Art of tunneling", "Manual on the planning and design of hydraulic tunnels" etc.

Sometimes, when a tunneling conditions warrant, a portal is required to be provided at the junction of the adit tunnel with the main tunnel in places where tunnel intersection and branching takes place. In such conditions, the load coming from the larger tunnel is required to be supported by the portal provided at the junction.

Tunnel portals are the important structures. They should be placed particularly at such locations where sound and stable rock faces are present in the uphill sides and gently sloping stable ground in the downhill side. They are often developed in the weathered or even unstable ground. Such situation happens when sound and strong rock exposures are absent either close to the desanding basin or around the forebay site. In selection of the inlet tunnel portal site, safety situation from the high flood should be addressed. Identification of the tunnel portal sites will be primarily on the basis of the detailed geological mapping and seismic survey or 2 D resistivity survey.

The most important functions of the tunnel portal are:

- To protect and support the adit, exits and approaches under masses of earth,
- To keep out and drain surface water running down the front slope,
- To emphasize the structural significance of the tunnel through architectural features.

From the structural point of view portals may be divided into the following categories:

- In solid rock there may be no need for a portal façade at all and left exposed at the approaches,
- If there is any danger of rolling rock or surface water seepage then a façade wall has to be provided around the entire opening,
- If there is earth pressure to be expected from the front slope then the portals have to be designed as retaining walls. Any retaining walls lining the approach cuts can be considered as counter forts supporting the portal.

Surface water running down the front slope should be intercepted behind the portal and drained so as to eliminate any seepage into the tunnel.

Drain Tunnel

The drain tunnel is to be situated at the end of the headrace tunnel. A drain pipe of an appropriate diameter with a hollow jet valve should be embedded in the concrete plug for releasing the water in the headrace tunnel while undertaking maintenance work. It will be appropriate to plan the drain tunnel at such location so that it can drain water safely into the nearest tributary stream.

Submergence and Air Vent

Entrance of air inside the tunnel is not desirable however, in certain conditions as follows air may enter and accumulate in a tunnel:

- During filling , air may be trapped along the crown at high points or at changes in cross sectional size or shape,
- Air may be entrained at intake either by vortex action or by means of hydraulic jump associated with gate opening,
- Air dissolved in the flowing water may come out of solution as a result of decrease in pressure along the tunnel,
- If Geometric approach in the tunnel is not appropriate,
- If sufficient submergence is not provided in the tunnel entrance.

Pressure tunnel must maintain suction head at its entrance to avoid air entering into it which creates an unnecessary complication for the smooth flow. The minimum suction head required is given by the formula (3.41) above in Section 3.1.1.4.

The presence of air in the pressure tunnel can be a source of grave damage as detailed below:

- The localization of an air pocket at the high point in a tunnel or at a change in slope which occasions a marked loss of head and diminition of discharge,
- The slipping of a pocket or air in a tunnel and its rapid elimination by an air vent can cause a water hammer,
- The supply of a mixture of air and water to a turbine affects its operation by a drop in output and efficiency thus adversely affecting the operation of the generator.

The following steps are recommended to prevent the entry of air in a tunnel.

- Intakes should be designed properly. A shallow intakes likely to cause air being sucked in the intake.

- Throughout the length of tunnel the velocity should remain constant or increase towards the outlet end
- Partial gate openings resulting in hydraulic jumps should be avoided
- Traps of pockets along high points and crown should be avoided.
- Thorough and careful surge analysis should be carried out to see that at no points on the tunnel section, negative pressure is developed.

If by any means air enters inside the pressurized tunnel, there should be a provision to release the air into the atmosphere at the earliest. An air vent pipe just downstream of the control gate at the tunnel entrance is one of the most effective means of releasing air from the pressure tunnel. The main functions of the air vent pipe are:

- Admission of air to nullify the vacuum effect which will be created when the water in the penstock drains after the intake gate is closed,
- The intake gates operate under condition of balanced pressure on both sides. For this purpose, the conduit is required to be filled through a bypass pipe. The entrapped air is, therefore, driven out through the air vent pipe.

Hence, it is necessary that an air vent pipe of adequate capacity is provided. The volume of air required per minute for the relevant flow conditions should be evaluated.

Design Basis for Tunnel Support System

The support system requirements for tunnel design are dependent on quality of rock mass through which tunnel is driven. In this connection, the parameters like Rock Mass Rating (RMR) values and the Rock Quality Index (Q) value have important bearing on choosing the tunnel support system. The following paragraphs deal with these parameters in respect of support system.

Rock Quality versus Support System

Tunneling for the 1 MW Tinau Hydropower Project done through the Siwalik rocks in the seventies was a pioneering activity to convince the concerned institutions and technicians that tunneling through the hills and mountains of Nepal is technically feasible. To date some 19 numbers of tunnels have been excavated in Nepal for the purpose of hydropower development and their construction involved different type of supporting works such as stone masonry, concrete, reinforced cement concrete, shotcrete with or without steel mesh and steel ribs.

Past study of the tunnel construction works in Nepal exhibits that the rock quality encountered differs from the types like excessively good / extremely good (Class – A) to exceptionally poor (Class – G) requiring the rock support types namely Class – I to Class – VI respectively.

Rock Mass Classification: The studies to be conducted on regional geology, surface geological mapping, geophysical survey, and exploratory drilling during the feasibility study stage of a tunnel provide a basis for very crude rock mass classification upon which preliminary estimate of a rock support requirement can be based. The information acquired during the preliminary design stage through the test aditing with supplementation of a few judiciously positioned boreholes will be more accurate and confirmative data for rock mass classification. The information collected should be useful to establish the Rock Mass Rating Value (RMR-value) and Rock Tunneling Quality Index, Q-value and consequently the required tunnel support is determined corresponding to the obtained RMR-value and Q-value.

The following six parameters are considered to classify a rock mass using the RMR system. The RMR value is determined by summation of the prescribed ratings corresponding to the respective values of the above parameters of the observed rock mass as given in Table 3.12.

- Uniaxial compressive strength of rock material,
- Rock Quality Designation (RQD),
- Spacing of discontinuities,

- Condition of discontinuities,
- Groundwater Conditions, and
- Orientation of discontinuities.

Based on the RMR value, the rock mass is categorized into the different classes and accordingly the rock support required for the preliminary tunnel design can be determined from the Tables 3.12 to 3.15 (Bieniaswki, 1989). To date, Tunneling Quality Index (Q) value is also used for determination of rock mass characteristics and the tunnel support requirement. The numerical value of this index Q is defined by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (3.74)$$

Where RQD is the Rock Quality Designation

J_n is the joint set number

J_r is the joint roughness number

J_a is the joint alternation number

J_w is the joint water reduction factor

SRF is the stress reduction factor

Table 3.12: Rock Mass Rating System (After Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of Values						
1.	Strength of intact rock material	Point-load strength index	> 10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range uniaxial compressive test is preferred		
		Uniaxial comp. strength	> 250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2.	Drill core Quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	< 25%		
	Rating		20	17	13	8	3		
3.	Spacing of discontinuities		> 2 m	0.6-2 m	200-600 mm	60-200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4.	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 m Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5.	Ground water	Inflow per 10 m tunnel length (1/m)	None	< 10	10-25	25-125	> 125		
		(Joint water press)/(Major principal б	0	< 0.1	0.1-0.2	0.2-0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100-81	80-61	60-41	40-21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300-400	200-300	100-200	< 100
Friction angle of rock mass (deg)	> 45	35-45	25-35	15-25	< 15

E. GUIDELINES FOR CLASSFICATION OF DISCONTINUITY CONDITIONS

Discontinuity length (persistence)	< 1 m	1-3 m	3-10 m	10-20 m	> 20 m
Rating	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1-1.0 mm	1-5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slicken sided
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Rating	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip-Dip 45^0-90^0	Drive with dip-Dip 20-45^0	Dip 45^0-90^0	Dip 20-45^0
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip-Dip 45^0-90^0	Drive against dip-Dip 20^0-45^0	Dip 0^0-20^0 Irrespective of strike^0	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al. (1972).

Table 3.13: Recommended rock support measures

Rock Support Class	Rock Mass Classification: Q	Rock Mass Classification: RMR	Rock Mass Classification: GSI	Mode of Failure	Support: Type	Support: Dimensions	Comments / Influence on Construction
I	Class A Excessively Good to Extremely Good > 100	Class I Very Good Rock 81 – 100	> 80	Generally Stable	No support	N/A	None
II	Class B Good to Very good 10 to 100	Class II Good Rock 61 – 80	55 to 80	Local rockfalls	Spot bolts	2.0 m dowels or longer to secure specific blocks	Immediate support where workers safety is impacted.
III	Classes D & C Poor to Fair 1 to 10	Class III Fair Rock 41 to 60	45 to 65	Loosening of the rock mass	Pattern bolting, straps and localized reinforced shotcrete	2.0 m dowels at 1.5 – 2.0 m spacing, 50 mm shotcrete in localized weak or unstable areas, Spot bolting as required	Support soon after mucking to prevent further relaxation.
IV	Class E Very Poor 0.1 to 1	Class IV Poor Rock 21 to 40	30 to 45	Progressive relaxation of the rock mass	Pattern bolting and fibre reinforced shotcrete, or shotcrete and mesh if adhesion is a problem	2.0 m dowels at 1 to 1.5 m spacing. 50 to 90 mm of shotcrete in crown	Support to be provided within one tunnel diameter of the face
V	Class F Extremely Poor 0.01 to 0.1	Class V Very Poor Rock 3 to 20	20 to 35	Local roof falls and deformation in jointed, weak or weathered rock	Pattern bolting and fibre reinforced shotcrete or shotcrete and mesh if adhesion is a problem	2.0 m dowels at 1m spacing. 90 – 150 mm shotcrete in crown and 50 to 90 mm on walls	Support to be provided immediately behind the exposed face
VI	Class G Exceptionally Poor < 0.01	Class V Very Poor Rock < 3	0 to 20	Potentially large roof falls due to weak fault zones, shearing or colluvium; significant deformation. Also zones with no self supporting capability and colluvium. Zero standup time.	Steel arches with lagging, shotcrete and rockbolts with reinforced ribs of shotcrete, or cast concrete lining. Stabilization such as forepoling, spilling and grouting may be required ahead of the face.	Up to 100 mm of quick setting shotcrete may be required as temporary support. Invert struts may be needed in highly stressed gound, or yielding ribs with friction joints.	Tunnel rounds to be shortened. Support to be provided immediately at and behind the face. Much handwork as support must be installed under the forepoling or grout canopy.

N/A denotes not available.

Table 3.14: Guidelines for excavation and support of 10 m span rock tunnels in accordance with RMR system (After Bieniawski, 1989)

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I – Very good rock RMR: 81-100	Full face, 3 m advance	Generally no support required except spot bolting		
II – Good rock RMR: 61-80	Full face, 1-1.5 m advance. Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
III – Fair rock RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown	50-100 mm in crown and 30 mm in sides	None
IV – Poor rock RMR: 21-40	Top heading and bench 1.0-1.5 m advance in top heading Install support concurrently with excavation, 10 m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
V – Very poor rock RMR: < 20	Multiple drifts 0.5-1.5 m advance in top heading, Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert	150-200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

The Tunneling Quality Index (Q) is determined using the values of the above parameters corresponding to the observed rock mass as given in the Table nos. 3.15(i) & (ii) around (Barton, et al, 1974). On the basis of the obtained Q-value of the rock mass, an appropriate rock support will be selected from the Table 3.13. Wherever the value of the index Q and Equivalent Dimension (D_e) are available, the Fig. 3.15 can be used for determination of required rock support type.

Similarly on the basis of the known Rock Structure Rating the amount of the shotcrete to be required in combination of either the rock bolt or different steel rib support can be determined. The figure 3.16 illustrates the general application of the above mentioned support for a tunnel of particular shape and size.

The rock support type selected on the basis of the RMR value and Tunneling Quality Index (Q-value) during the preliminary design stage usually requires modification while undertaking the construction works as a result of encountering variation in rock quality from the anticipated ones either as the better or poorer ones.

Table 3.15(i): Classification of Individual Parameters used in the Tunneling Quality Index Q (After Barton et al, 1974)

1. Rock Quality Designation			**RQD**
A	Very Poor		0 – 25
B	Poor		25 – 50
C	Fair		50 – 75
D	Good		75 – 90
E	Excellent		90 – 100
Note:	i) Where RQD is reported or measured as $\leq$10 (including 0), a nominal value of 10 used to evaluate Q. ii) RQD intervals of 5, i.e., 100, 95, 90, etc, are sufficiently accurate.		
2. Joint Set Number			**J_n**
A	Massive, no or few joints		0.5 – 1.0
B	One joint set		2
C	One joint set plus random joints		3
D	Two joint sets		4
E	Two joint sets plus random joints		6
F	Three joint sets		9
G	Three joint sets plus random joints		12
H	Four or more joint sets, random, heavily jointed, "sugar cube", etc		15
J	Crushed rock, earthlike		20
Note:	i) For intersections, use (3.0 X J_n) ii) For portals, use (2.0 X J_n)		
3. Joint Roughness Number			**J_r**
a) Rock-Wall contact, and b) rock-wall contact before 10 cm shear			
A	Discontinuitous		4
B	Rough or irregular, undulating		3
C	Smooth, undulating		2
D	Slikensided, undulating		1.5
E	Rough or irregular, planar		1.5
F	Smooth, planar		1.0
G	Slickensided, planar		0.5
Note:	i) Descriptions refer to small scale features and intermediate scale features, in that order.		
c) No rock-wall contact when sheared			
H	Zone containing clay minerals thick enough to prevent rock-wall contact		1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact		1.0
Note:	i) Add 1.0 if the mean spacing of the relevant joint set is greater that 3 m. ii) J_r = 0.5 can be used for planar slicksided joints having lineations, provided the lineations are orientated for minimum strength.		
4. Joint Alteration Number		ϕ_r approx	J_a
A	Tightly healed, hard, non-softening, impermeable filling, i.e. quartz or epidote		0.75
B	Unaltered joint walls, surface staining only	25 – 35^0	1.0
C	Slightly altered joint walls, Non-softening mineral coatings, sandy particles, clay-free, disintegrated rock, etc.	25 – 30^0	2.0
D	Silty – or sandy-clay coatings, small clay fraction (non-softening)	20 - 25^0	3.0
E	Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chorite, tale, gypsum, graphite, etc. and small quantities of swelling clay	8 - 16^0	4.0
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc	25 – 35^0	4.0
G	Strongly over-consolidated non-softening clay mineral fillings	16 - 24^0	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but < 5 mm thickness)	12 – 16^0	8.0
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but < 5 mm thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water, etc.	6 – 12^0	8 - 12
c) No rock-wall contact when sheared (thick mineral fillings)			
KLM	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6 – 24^0	6, 8 or 8 -12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	-	5.0
OPR	Thick, continuous zones of bands of clay (see G, H, J for description of clay condition)	6 – 24^0	10, 13 or 13-20

Table 3.15(ii): Classification of Individual Parameters used in the Tunneling Quality Index Q (After Barton et al, 1974)

5. Joint Water Reduction Factor		**Approx. water pressure (kg/cm^2)**	**J_w**
A.	Dry excavations or minor inflow i.e., < 5 l/min locally	< 1	1.0
B.	Medium inflow or pressure, occasional outwash of joint fillings	1 – 2.5	0.66
C.	Large inflow or high pressure in competent rock with unfilled joints	2.5 – 10	0.5
D.	Large inflow or high pressure, considerable outwash of joint fillings	2.5 – 10	0.33
E.	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2-0.1
F.	Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1-0.05
Note:	i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed ii) Special problems caused by ice formation are not considered.		

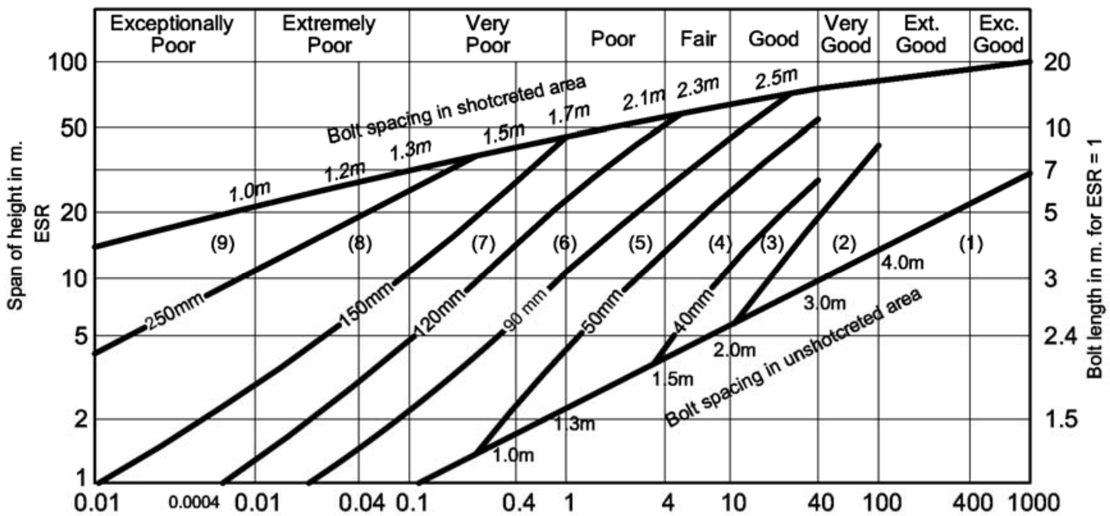
6. Stress Reduction Factor		**SRF**
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.		
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leq$ 50 m)	5
C.	Single weakness zones containing clay or chemically disintegrated rock (depth of > 50 m)	2.5
D.	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
E.	Single shear zones in competent rock (clay-free))(depth of excavation $\leq$ 50 m)	5.0
F.	Single shear zones in competent rock (clay-free) (depth of excavation > 50 m)	2.5
G.	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)	5.0
Note:	i) Reduce these values of SRF by 25 – 50% if the relevant shear zones only influence but do not intersect the excavation.	

b) Competent rock, rock stress problems		σ_c/σ_1	σ_0/σ_c	SRF
H	Low stress, near surface, open joints	> 200	< 0.01	2.5
I	Medium stress, favourable stress condition	200-10	0.01-0.3	1.0
J	High stress, very tight surface. Usually favourable to stability, may be unfavourable for wall stability	10-5	0.3-0.4	0.5-2
K	Moderate slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
L	Slabbing and rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
M	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	< 2	> 1	200-400
Note:	ii) For strongly anisotropic virgin stress field (if measured): when 5 $<\sigma_1/\sigma_3<$ 10, reduce σ_c to 0.7 σ_c. When $\sigma_1/\sigma_3>$ 10, reduce σ_c to 0.5 σ_c, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_0= maximum tangential stress (estimated from elastic theory). iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 such cases (See H).			

c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure		σ_0/σ_c	SRF
N	Mild squeezing rock pressure	1 - 5	5 - 10
O	Heavy squeezing rock pressure	> 5	10 - 20
Note:	iv: Cases of squeezing rock may occur the depth H > 350 $O^{1/3}$ (Singh et al., 1992). Rock mass compression strength can be estimated from $q = 0.7\gamma O^{1/3}$ (MPa) where γ =rock density in kN/m^3 (Singh, 1993).		

d) Swelling rock: chemical swelling activity depending on presence of water		
R	Mild swelling rock pressure	5 – 10
S	Heavy swelling rock pressure	10 – 15
Note:	J_1 and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, τ (where $\tau \sim \sigma_n \tan^{-1}\left(J_r / J_n\right)$. Choose the most likely feature to allow failure to initiate.	

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$



REINFORCEMENT CATEGORIES
1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40-100 Unreinforced shotcrete
5) Fibre reinforced shotcrete 50-90 mm and bolting
6) Fibre reinforced shotcrete 90-120 mm and bolting
7) Fibre reinforced shotcrete 120-150 mm and bolting
8) Fibre reinforced shotcrete > 150 mm, with reinforced Ribs of shotcrete bolting
9) Cast concrete lining

Fig. 3.15: Estimated support categories based on the tunneling quality index *Q*. (After Grimstad and Barton, 1993)

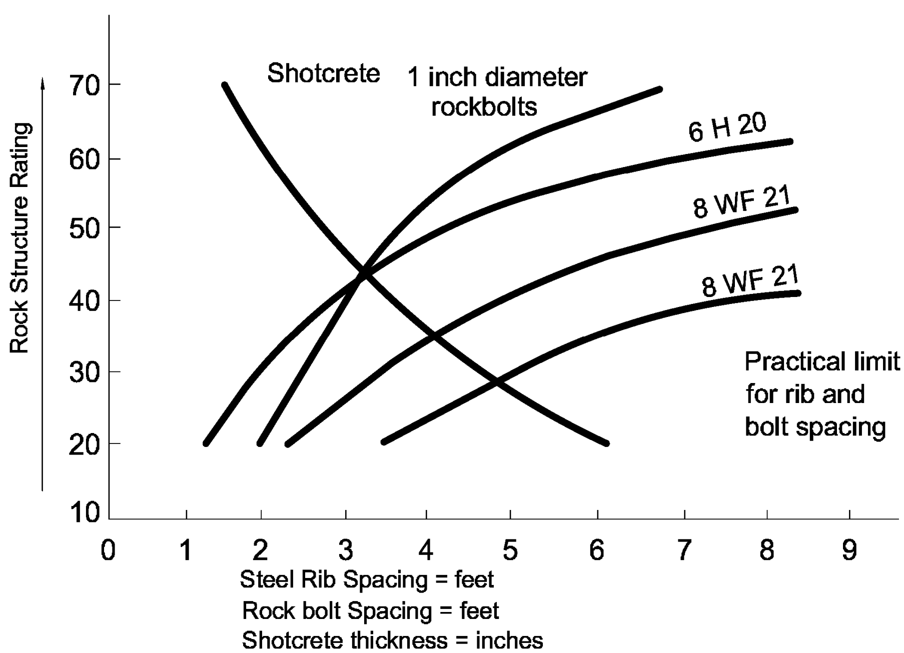


Fig. 3.16: RSR support estimates for a 24 ft. (7.3 m) diameter circular tunnel. Note that rockbolts and shotcrete are generally used together. (After Wickham et. Al., 1972)

Adit Plug: After completion, an adit plug is required to be constructed. It should be supplemented by the cement grouting of the surrounding rocks to seal the water conveyance system. A steel bulk head door will be installed to provide permanent access for maintenance. In addition an appropriate flushing pipe will be fitted with a valve for emptying of the tunnel within 24 hours. The grouting works will be accomplished using the following four types of grouting:

- Back-fill grouting to fill spaces between initial lining and rock;
- Contact grouting to fill gaps between initial lining and plug concrete;
- Consolidation grouting to improve the quality of the surrounding rock, and
- Curtain grouting to form a zone of low permeability.

Rock Bolt and Shotcrete: When rock mass can withstand induced stresses, support is used to prevent the block from falling. Large blocks are normally supported by rock bolts, while shotcrete is used to prevent dissolution / disintegration / weathering of small block between rock bolts. Thus, bolts and shotcrete act together as a combined support system.

Shotcrete is applied not only to prevent disintegration of small rock block between rock bolt but also to hold together intact. Uses of shotcrete function on two principles:

- Adhesion between rock and shotcrete, and
- Arching effect of a shotcrete layer

Shotcrete can follow the rock surface forming a combined rock shotcrete structure which can withstand remarkably induced stress. Shotcrete is used immediately after the excavation. It is in escalating use since it minimizes the labour intensive process of mesh installation than applying rock bolt and mesh support in the underground excavations.

Current shotcrete support design perception relies much upon rules of thumb and precedent experience. The thickness of a shotcrete tunnel lining is ascertained in relation to (i) Rock Structure Rating as given in Figure 3.16 (Wickham et at, 1972), (ii) Rock Mass Rating as shown in Tables 3.13 & 3.14 (Bieniawski, 1989) and (iii) Tunnel Quality Index Q as presented in Figure 3.15 (Grimstad and Barton, 1993). A compilation of a current shotcrete practice is also available as given in Tables 3.15(i) & (ii). The above mentioned figures and tables can only be used as an approximate guidance while making decision upon type, thickness and quality of shotcrete to be considered in a specific application. Modification will be certainly required in conformity with the local variation in rock condition.

Shotcrete can assist in controlling deformation, particularly when applied in combination with rock bolts or dowels or cables. It can not prevent deformation from taking place, especially in high stress condition. Its use becomes very effective when bolt or cable installations are carried out after an initial shotcrete application. Such consideration allows transmission of the fan plate loads over a large area to underlying rock mass.

Concrete Lining: The concrete lining is used in tunnels designed for long life, irrespective of immediate need for support. It is particularly preferred at the tunnel portals, siphon & aqueducts and in the sections of the insignificant overburden thickness. The concrete lining can also be done in combination of timber support or steel line support. Minimum thickness of concrete lining is considered to be 15 mm which increases towards the arch.

Steel Ribs: Steel ribs are commonly used in for wide openings and semi-permanent structure such as underground sub-station, large dimension tunnels. Steel rib support is invariably installed to the face shortly after blasting a round. Such support is considered either to provide passive resistance to the load exerted by the rock or to act as a system of reinforcement.

Grouting: For waterway tunnels through jointed rock or through which seepage is to be minimized, the surrounding rock of the tunnel lining are usually grouted in order to:

- Consolidate rock materials;
- Fill open fissures / spaces in the rock; and
- Fill gap between lining and rocks.

Consequently four types of grouting are carried out for the waterways and related works such as back fill, consolidation, curtain and contact grouting. The pressure selection for grouting (4 – 20 kg/cm^2) should be taken into account in consideration of objective of grouting purpose and geological condition. The objectives of different types of grouting are as follows:

Back fill grouting is done to fill the gaps between the concrete lining and the surrounding rocks, at top where concrete is difficult to spread out thoroughly.

Consolidation grouting is performed for consolidating the surrounding rocks and filling open fissures in the rock.

Curtain Grouting has a purpose of preventing water seepage from the waterway end portion (tunnel portals, Adit plug) towards ground surface. As a result, a wide range of low permeability is formed.

Contact Grouting is applied to fill the gaps between the concrete lining and the concrete plug so as to ensure continuous contact.

Protection Works: Surface protection work may be required at the portals of the headrace tunnel or drain tunnel or tailrace tunnel or at the entrance of the adits and surge tank / surge shafts particularly under the situation when the encountered rock was highly weathered one or the actual rock line was observed greater than anticipated during the investigation stage. The required slope protection works will be either stone pitching or rubble masonry or shotcrete or shotcrete with bolts or concrete facing or concrete gravity wall.

Instrumentation: Instrumentation in tunneling is generally practiced before, during and after the construction of tunnel. Instrumentation during the design stage or before the construction of an underground excavation is aimed to acquire the information on the design data such as:

- Modulus of deformation of the rock mass;
- Strength of the in-situ rocks, and
- In-situ state of stress.

In-Situ Deformation Modules: On the basis of historical case studies, back analysis of measured deformation and prediction from numerical analysis, the following equations have been established for calculation of the deformation module (E_m). This modulus is required for numerical studies of stress and displacement distribution around underground excavations.

$E_m = 2RMR - 100$ (Bieniawski, 1978) ------------------------------------ (3.75)

$E_m = 10^{\frac{(RMR-10)}{40}}$ (Serafin and Pereira, 1983) --------------------------------- (3.76)

$E_m = 25\log 10^{Q}$ (Grimstad and Barton, 1993) ---------------------------- (3.77)

The Figure 3.17 illustrates the comparison of fitness of the equation for prediction of in situ-deformation modulii.

In-Situ Rock Strength Testing: Many methods for measuring in-situ stress are available. The most common techniques are namely:

- Hydraulic fracture techniques
- Direct stress measurement using flat jacks, and
- Borehole method of stress measurement.

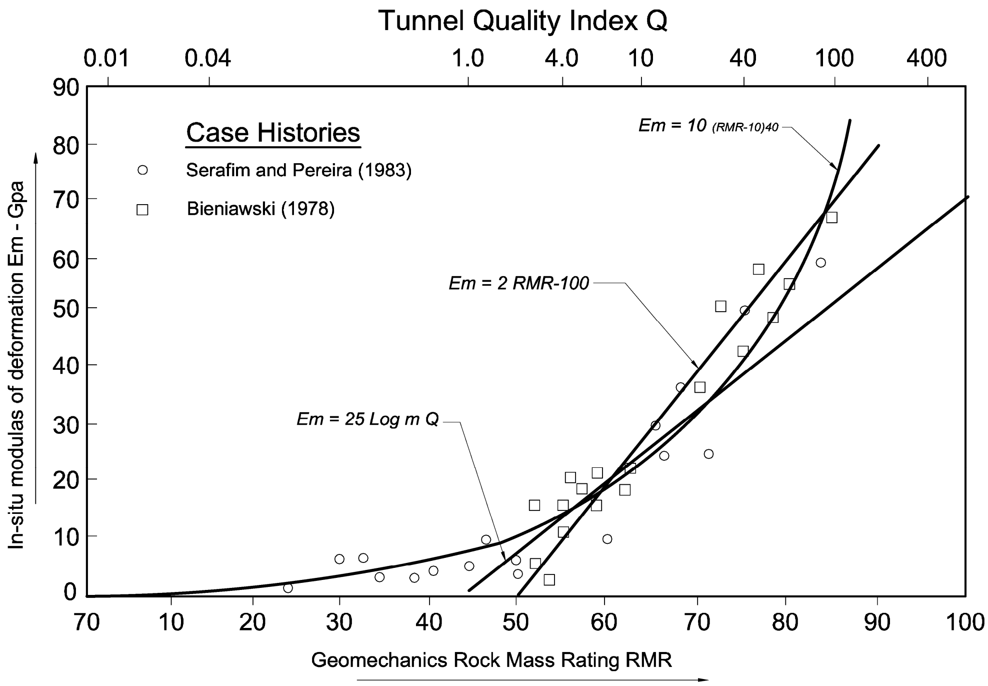
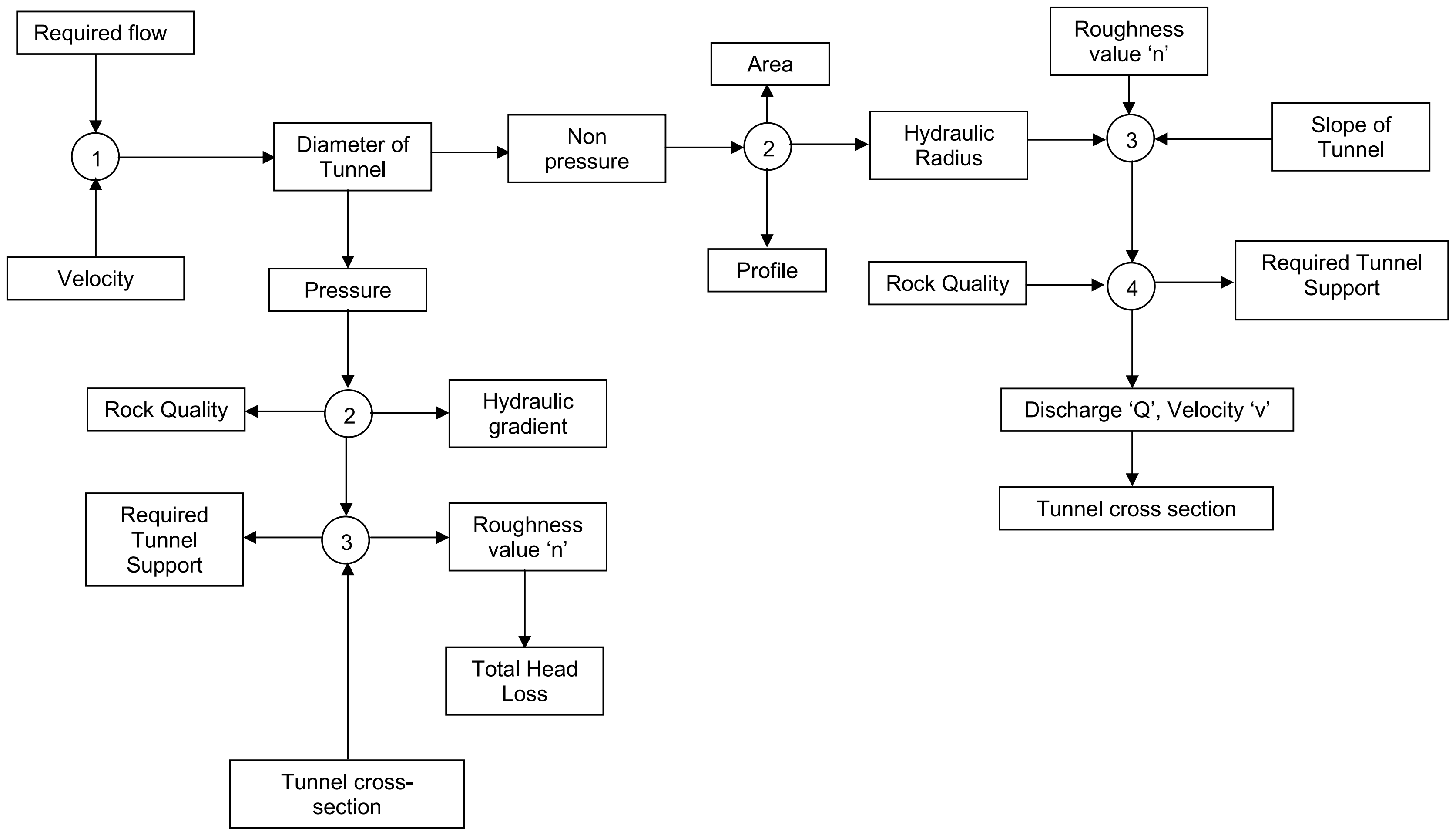


Figure-3.17: Prediction of in-situ deformation modulus *E*m from rock mass classifications.

Service, Operation and Maintenance: Generally, all tunnels are rather special and delicate types of structures from the point of view of design as well as construction. Because of restricted clearances, poor visibility and the continuous destructive action of natural forces there is special need for careful maintenance, regular inspection and rather comprehensive renovation and reconstruction. The designer should also provide guidelines for service, operation and maintenance of the proposed tunnel.

The flow chart for the design of tunnel is shown in Chart 2 below.

Chart 2: Flow Chart for Tunnel Design



3.1.1.6 Cross Drainage Works

General

The water conveyance system (canal or pipe) laid down on a ground surface or buried for hydropower development have to cross generally the streams, drains and sometimes road, railway, valley or depressions, the structural works necessary to be incorporated in the water conveyance system for such crossings are called cross drainage works. They include aqueducts, siphons and flumes. The design guides for these cross drainage works are briefly described in the following sub-sections.

Aqueducts

An aqueduct literally means a channel for conveying water and it may be either above or below the ground. In water resources engineering the term is confined to mean a structure carrying canal over a drainage channel without having to lower down the bed of the drainage channel for the crossing. Hydraulic designs of the aqueducts are similar to the open canal using Manning's formula. A typical section of the pipe aqueduct is shown in Fig. 3.18.

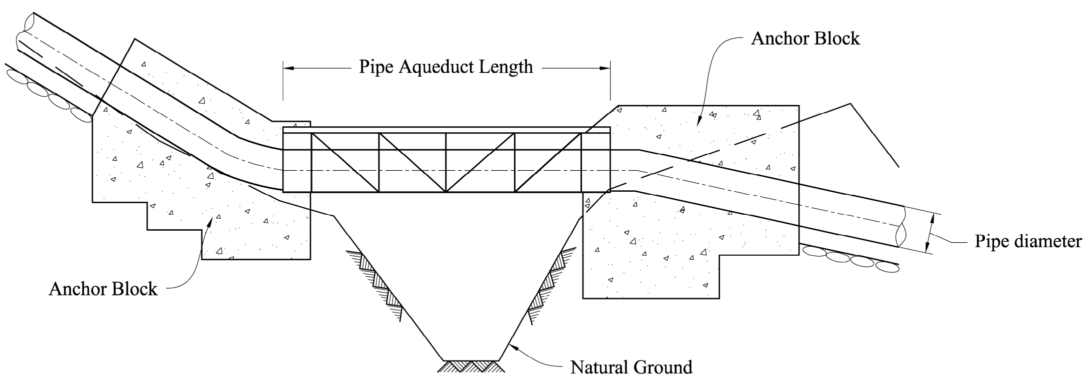


Fig. 3.18: Pipe Aqueduct

Siphons

Siphons are usually proposed to cross a large drainage channel, rivers along the canal alignment. It is usually more economical to carry the canal water under the channel in an inverted siphon than to carry the drainage water under the canal through a culvert. Siphon provides excellent reliability, as the accuracy of the cross drainage flow predication is less critical where siphons are used. However, the use of siphon is contingent upon availability of head for siphon losses. Other factors which will affect the results of a cost comparison are the width and depth of the drainage channel. A typical section of a pipe siphon is shown in Fig. 3.19.

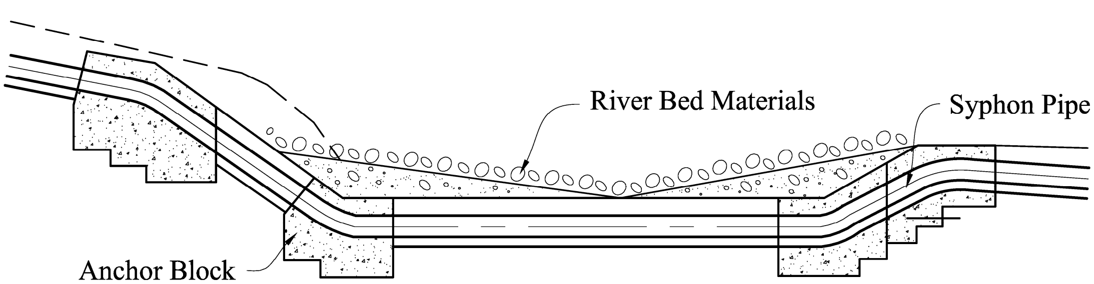


Fig. 3.19: Typical Section of Pipe Siphon

Hydraulic Consideration: Available head, economy and allowable pipe velocities determine the size of the siphon pipe. Thus it is necessary to assume internal dimensions for the siphon and compute head-losses such as entrance, friction, bend, exit and trashrack. The sum of all the computed losses should approximate the difference in energy grade elevation between the upstream and downstream ends of the siphon. Adequate submergence below upstream water level should be provided to conduit at the inlet.

Design Procedure:

- Determine the structures required at inlet and outlet and size of pipe
- Compute siphon head losses in the layout. If the head losses do not agree with the available head, it may be necessary to make some adjustment such as pipe size, etc.

If the computed head losses are greater than the available head, i.e. greater than the difference in upstream and downstream water level, the siphon will cause backwater effect upstream. In this case either the size of the pipe has to be increased to reduce the head losses or revised the canal profile. If the computed head losses are appreciably less than the difference in upstream and downstream water surface, it may be possible to decrease the size of pipe or the canal profile may be revised so that the available head is approximately the same as the head losses

Flumes

Flumes are used to convey canal water along steep side hill terrain, or to convey canal water over other waterways, or natural drainage channels. Flumes are also used at locations where there is restricted right of way or where lack of suitable material makes construction of canal banks undesirable or impracticable. Flumes supported on a bench excavated into a hillside are called bench flumes and flumes supported above the ground with reinforced concrete, structural steel, or timbers are called elevated flumes. Flumes are hydraulically designed as an open canal flow using Manning's formula as described above.

3.1.1.7 Structural Elements to be Incorporated in the Water Conveyance System for Controlling Water-Hammer

General

The sudden closure of the terminal closing mechanism, wicket gate or valve guarding the turbines is followed by inertia effects resulting in over pressures and depressions in the penstock or pressure conduit, which should be allowed too in the structural design. Such pressure changes induce the effect of water-hammer in the pressure conduit. During load demand, low-pressure waves are formed at the turbine as the water column is accelerated. This could result into water column separation creating vacuum zone, subsequent refilling of this vacuum zone or rejoinder of the water columns can create extremely high pressure.

There are number of ways for controlling water-hammer. They are listed below:

- Reduce penstock velocities (enlarge conduit diameters);
- Reduce length of waterways. Profile changes can also alleviate some problems;
- Reduce valve closure times or opening times;
- Vary machine hydraulic characteristics;
- Increase WR^2 (a constant called flywheel effect);
- Change wave velocity;
- Install pressure control valves;
- Add a surge tank;
- Add a cushioning stroke on the turbine (two closure rates);
- Add air chambers.

Since the present study is concerned only with the design of water conveyance system, the methods of controlling water-hammer related to the structural elements to be incorporated in the water conveyance system such as forebay and surge tank / shaft only have been dealt with here.

Forebay (Headpond)

The forebay (headpond) is essentially a broadened section of the canal in which a gated spillway is installed. The purpose is to distribute evenly, over a proper transition, the water conveyed by the power canal among the penstocks and, at the same time, to regulate the water flow into the latter, as well as to ensure the disposal of excess water. At the forebay sediment still carried by the water settles down. The storage capacity of headpond tends also to reduce the drop of water-level in case of sudden load increase. Headponds having a great storage capacity may even provide daily pondage for the plant. Thus the forebay acts as a regulating pondage to cushion the impact of sudden load rejection or load acceptance.

The upsurge in the forebay caused by the load rejection is estimated using following equation.

In case of sudden closure, the maximum height of the surge is given by the expression known as:

E. Feifel's equation:

$$\Delta h_{\max} = \frac{V^2}{2g} + \sqrt{\left(\frac{V^2}{2g}\right)^2 + 2\frac{V^2}{2g} \times h} \qquad \text{--------------------------------} (3.78)$$

For the gradual and complete closure

$$\Delta h_{\max} = \frac{V^2}{4g} + V\sqrt{\frac{h}{g}} \qquad \text{----------------------------} \quad (3.79)$$

where

V = water velocity (mean velocity of flow)

h = water depth – effective depth

$\Delta h_{\max}$ = height of the surge

In general total capacity of the forebay is fixed so that the total live storage volume in it is equivalent to the total volume required for three minutes of operation. Three minutes of operation is adopted from the criteria of realistic water starting time, which is in the order of 1 to 2 minutes and an extra minute for the forebay. Realistic water starting time is the time required accelerating the water in the water conductors from rest to the steady state velocity at full gate discharge with all units on a given penstock of power conduit in operation*.

A schematic sketch of a forebay showing plan, profile and sections is presented in Fig. 3.20 and the flow chart for the design of forebay is shown in Chart 3 below.

* Reference: Davi's Handbook of applied Hydraulic- fourth edition

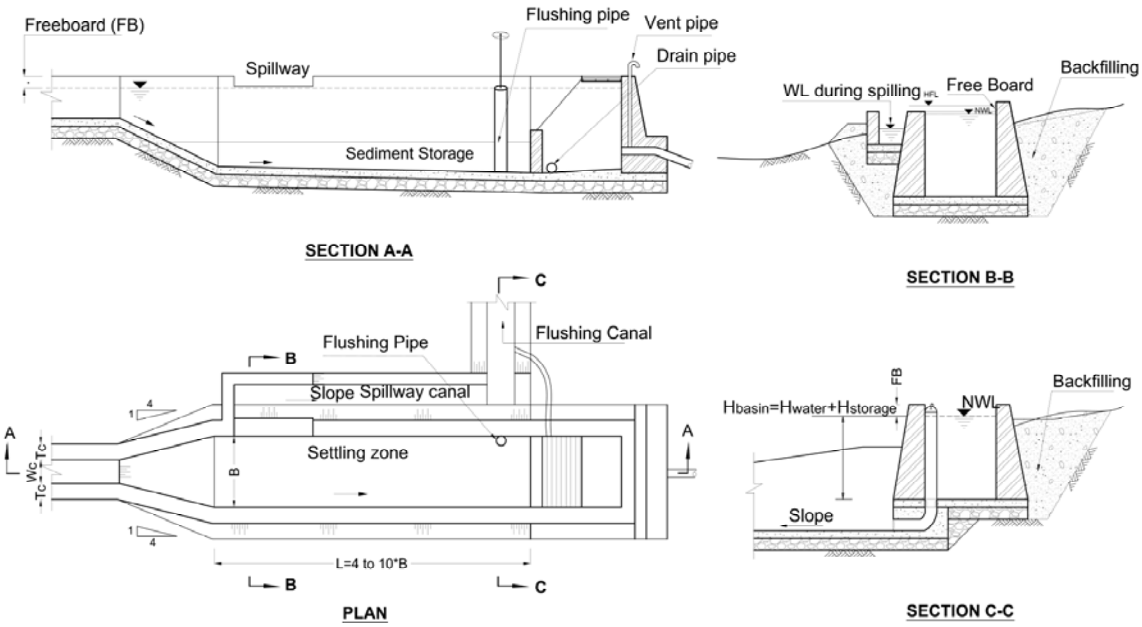


Fig. 3.20: Forebay Plan and Profile with Cross-section

Pressure penstock pipe starts from the end of the forebay chamber. The portion of forebay from where the penstock starts often called as a penstock chamber. A sill between the penstock bottom part and the penstock chamber is maintained which is often half a meter. The upper part of the penstock is located with sufficient suction head with the water surface in the chamber.

The suction head requirement is calculated using the formula (3.41).

Spillway releasing excess water from forebay to some natural channel is designed using following formula:

$$Q = CLH^{\frac{3}{2}} \qquad \text{------------------------------------} \qquad (3.80)$$

Where,

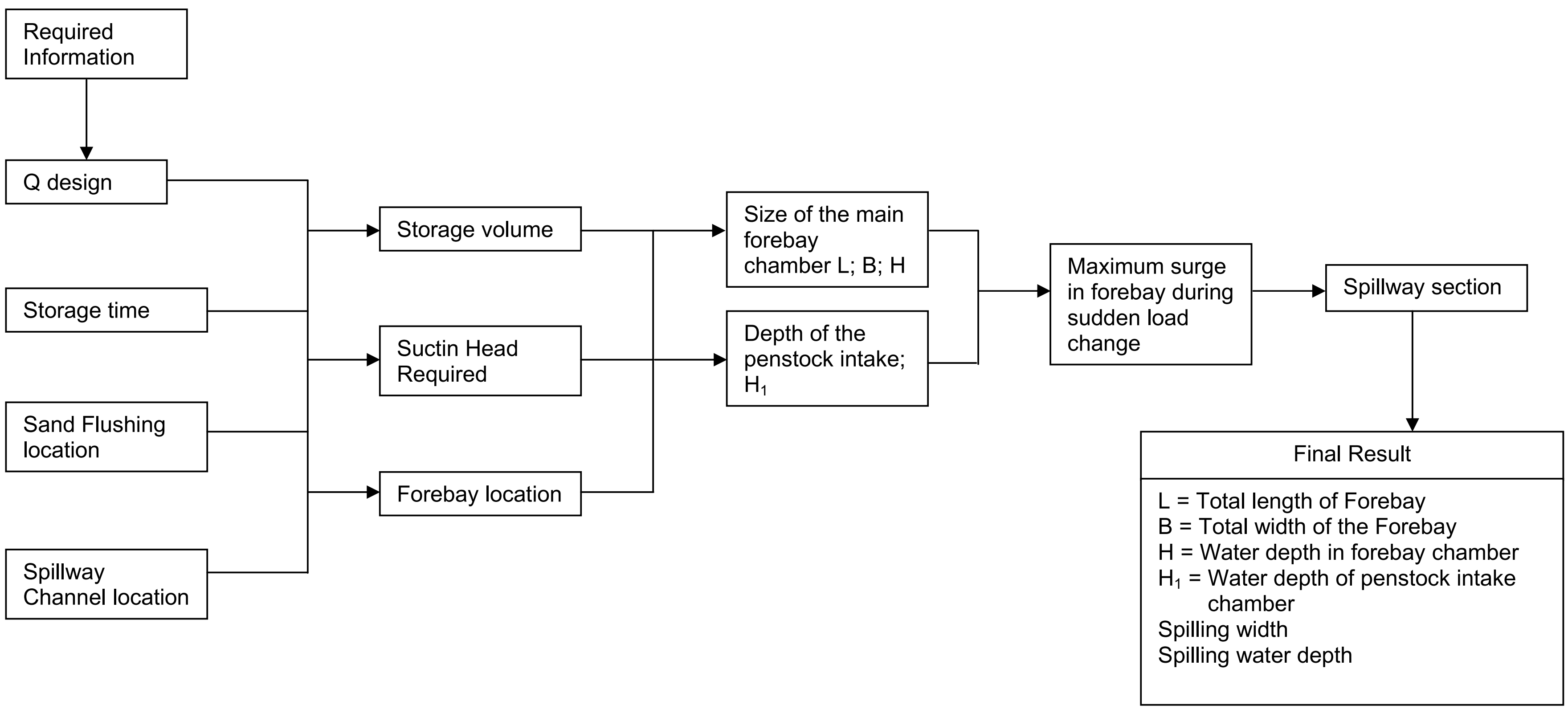
Q - Discharge in m^3/s

C - Discharge Coefficient depends on the shape of the structure

H - Water head in meter

L - Spillway Length

Chart 3: Flow Chart for Forebay Design



Surge Tank

General

Surge tank is located between the headrace pressure conduit and the steeply sloping penstock pipe and is designed either as a chamber excavated in the mountain or as a tower raising high above the surrounding terrain. Surge tank requirements are established from considerations of the conduit length and profile, velocity, WR^2 (Product of the revolving parts of the unit and the square of the radius of gyration -turbine runner, shaft and generator rotors), economics and operating requirements relating to governing and stable operation. In some instances surge tanks are applied both upstream of the penstock and downstream of the powerhouse. In any hydropower projects where the water is brought to the machines by a long pressure conduit, considerable inertia effects arise from the large mass of water in motion. This mass is of such magnitude that considerable force is necessary to accelerate or retard it. Moreover, water hammer effects will result even from partial load changes if the resulting turbine guide vane movements are at all rapid, as in fact they must be if undesirable speed rise is to be prevented. The pressure rises resulting from water hammer can be limited by the use of relief valves or similar appliances, but though these appliances will limit pressure rises effectively on reduction of load; i.e. on closure of the turbine guide vanes, they can not assist in accelerating the water column on increase of load, i.e. opening of the guide vanes. Where the pressure conduit is of considerable length, therefore, it frequently becomes desirable to introduce a surge chamber at a suitable point along the pressure conduit system.

Purpose of Surge Tank

The main purposes of surge tank are:

- Upon the rapid closure of the turbine in case of load rejection, water masses moving in the pressure tunnel and in the penstock are suddenly decelerated. Owing to the inertia of moving masses, high overpressures develop at the lower end of the penstock, which are propagated upwards in the penstock in the form of pressure wave. The magnitude of the so called water hammer, caused by decelerating the moving masses by closure, will depend upon the dimensions and elastic properties of the conduit and owing to the oscillating character of the phenomenon, the overpressure at any point along the conduit will be determined to a significant degree also by the time duration which closure is effected. The overpressure due to water hammer travels along the closed conduit and is not relieved until a free water surface is reached. An important function of a surge tank can already be derived from the foregoing: the closed pressure conduit connecting the power station to the reservoir is expediently interrupted by the surge tank to intercept the pressure wave due to water hammer at the free water surface and to except thereby the pressure tunnel from excessive pressure. The afore-mentioned function of the surge tank is by no means negligible, even if synchronous bypass relief valves are installed as a means of protection against excessive water hammer for the case of sudden load rejection. Rapid change in load, that does not result in total closure of the regulating mechanism and are insufficient to actuate the relief valve, may induce significant supernormal pressure attaining from 5 to 15 percent of the total head. Though penstocks can be designed without difficulty to resist similar overpressure, it would be extremely uneconomical to design the almost horizontal tunnel located relatively high, for overpressures amounting even to small fractions of the total load. It should be noted that surge tanks do not always prevent supernatural pressures from entering into the pressure tunnel. The pressure wave is but incompletely reflected from the surge basin and part of the pressure may be propagated further into the tunnel. The magnitude of the un-reflected part of the pressure wave depends upon the spatial arrangement of the junction of the tunnel, the surge tank and the penstock and to an even greater extent, upon the relative value of the cross sectional area of the communicating orifice. The pressure wave traveling up the tunnel may, in general, be neglected in designing the tunnel if a simple surge chamber having an orifice of ample size is applied. However, in case of so called restricted orifice surge tanks the un-reflected part of the pressure wave may be too significant to be neglected.

- The surge tank provides protection to the penstock against the detrimental effects of water hammer if no bypass valve is installed or if the bypass valve fails to operate. The effect of the surge in this case consists in reducing water hammer. Partial or total closures occurring during turbine operation are not instantaneous but requires even in the most unfavorable cases, several seconds. The pressure wave following the initial moment of valve closure travels with high velocity the entire length of the penstock. Thereupon a negative pressure wave traveling downstream with identical velocity is created, since the free water surface in the surge chamber functions as a reflecting surface. The overpressure, steadily increasing during the time of closure, is counterbalanced by the steadily increasing depression and thus the overpressure remains below the value occurring in case of identical, yet instantaneous closure. The maximum overpressure originated depends, beside other factors, upon the length of the penstock and is proportionate to it. Namely, the shorter the penstock the sooner the pressure wave reflects, consequently the lower the overpressure to be taken care of. The surge tank, by interrupting the closed system of the penstock and of the pressure tunnel, materially reduces the overpressure due to water hammer.*

- One of the most important purpose of surge tank is to provide water supply to the turbines in case of starting up (load demand) until the conduit velocity has accelerated to the new steady state value. When the turbine is started, water demand suddenly increases the flow of water in the penstock is suddenly initiated and respectively accelerated. In the steeply inclined penstock water is supplied as required by the demand, and the flow to the turbine remains continuous if the rate of opening is not too sudden, i.e. the opening time is not too short. On the other hand, owing to its inertia, the acceleration of the water mass in the almost horizontal pressure tunnel is significantly lower, and thus the flow around the elbow would become discontinuous if no surge tank were applied. The amount of water required during these changes in operating conditions is supplied by the surge tank installed in the conduit. The capacity should, therefore, be selected to ensure the required water supply during the most unfavorable increase in demand anticipated, until the water mass in the tunnel has attained the necessary velocity. To this end the water level in the surge tank must drop considerably to create a gradient from the reservoir to the chamber capable of accelerating the water mass moving in the tunnel. Air should be prevented from entering the penstock even in case of the deepest down surge in the chamber, moreover, with regard to the developing vortices; ample water cover should be maintained.

  The height of the surge tank is governed by the highest possible water level that can be anticipated during operation. The variations in the demand initiated by a rapid opening or closure of the terminal valve or turbine are followed with a time lag by the water masses moving in the tunnel. Upon the rapid and partial closure of the guide vanes following a sudden load decreases, water mass in the penstock are suddenly decelerated corresponding to the reduced power demand and one part of the continuous supply from the tunnel tends to fill the surge tank. Although the differential head between reservoir and surge tank is thus reduced and the flow in the tunnel is retarded, the water surface in the surge chamber will be raised to above the static level. The counter-pressure created by this over-travel decelerates the flow in the tunnel to an extent that the supply becomes smaller than the turbine demand and even counter-flow may occur. Consequently, the water surface in the chamber will start to recede and will drop to below the steady state level. In an attempt to establish steady flow conditions, the water surface will again start to rise from the low point, but owing to the inertia of moving water, will again over travel the steady state level. The cycle is repeated all over again, however, with amplitudes reduced by friction, i.e. the oscillation is damped. The phenomenon is the water surface oscillation. The maximum amplitude of water surface oscillation can be observed when due to some extraordinary reason, full load is suddenly rejected and the water demand is suddenly stopped.

Behaviors of the Surge Chamber

The motion of the water surface in a surge chamber is complex and it is influenced by many factors, notably by the physical size and nature of the pressure aqueduct system, by the method of governing the turbines, by the extent of the load changes which are to be accommodated, by the rate at which these changes occur, and by the physical characteristic of the chamber itself.

If a turbine is running at a steady load, water level in the surge chamber will be lower than that in the reservoir, the difference in level representing the velocity head and the head necessary to overcome friction and other losses. On abrupt reduction of the electrical load the turbine governor, in its efforts to maintain steady synchronous speed, will rapidly cause the turbine guide vanes to close, so that there will be an abrupt reduction of flow. This will initiate water hammer waves, which will be reflected to

* Water hammer is explained in the Penstock chapter.

and fro between the turbine and the surge chamber, and also an oscillation or surge of the entire mass of water in the pressure conduit and chamber. The water hammer effects are of very short duration and will die away before the surge has reached significant proportions. It is usual, therefore, to regard the surge of the water hammer as separate phenomena and to disregard the water hammer effects when evaluating surge. The surge causes the water level to rise in the chamber until exceeds reservoir level and produces a retarding force which will arrest and then reverse the direction of flow in the conduit. The chamber water level will then drop until it is below reservoir level; the conduit flow is again slowly stopped and reversed, and the cycle is repeated until damped out by frictional losses. The cycle is shown in the Figure 3.21 below.

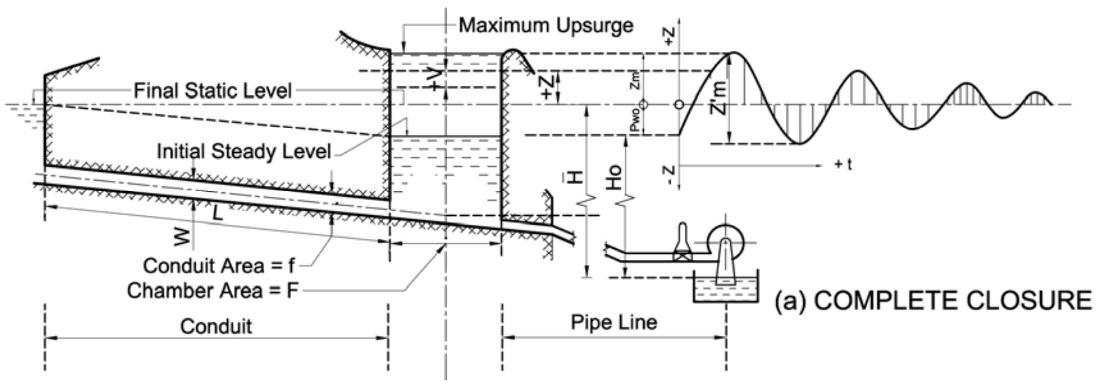


Fig. 3.21: The Surge Cycle under Complete Closure

On starting up the turbine with water at rest in the tunnel, the water level in the chamber will fall rapidly below its initial position at reservoir level, and will thus create sufficient head to accelerate the main mass of water in the tunnel. The head required to do this is greater than necessary to maintain steady flow against normal conduit losses. The conduit discharge will thus increase until it exceeds that required by the turbine; the surplus will cause a rise in chamber level, increasing the net head on the turbine and reducing its demand for water. The rise of chamber water level will retard the conduit flow and, as with load rejection, a surge motion will be set up and will continue until damped out by friction. The water level in the chamber will settle finally at its steady running level which, owing to friction and velocity effects, is lower than reservoir level. The damping effect is shown in the Figure 3.22 below.

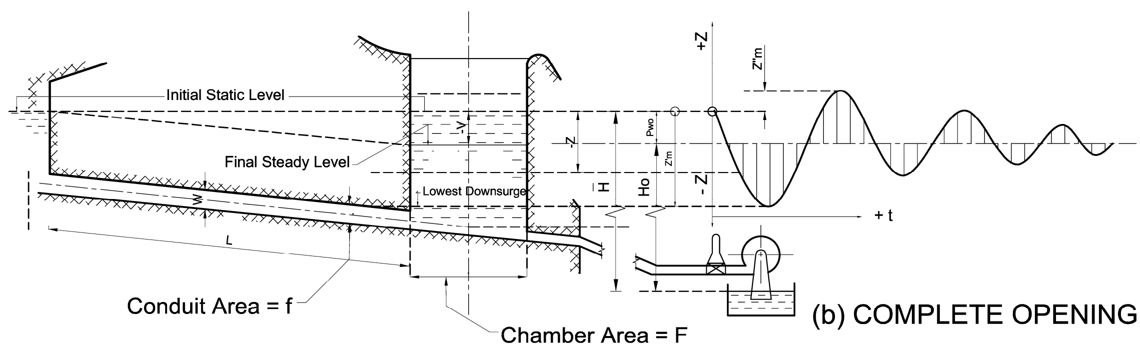


Fig. 3.22: Damping Effect under Complete Opening

H_o = Net head between chamber and turbines under initial steady

$\overline{H}$ = Net head between turbine centerline and final static level

P_{wo} = Total head loss between upstream water level and chamber

t = time interval

$\pm Z$ = up-surge and down-surge

Z_m = maximum or positive upsurge

Z'_m = minimum or negative surge
F = Chamber area
f = Conduit area

When partial load change occurs whether the load is increased or decreased, the behavior of the water in the surge chamber will be similar to that which occurs on full load change. For a given chamber, the surge resulting from partial load will be similar to those resulting from the full load change but of smaller amplitude. Throttled chambers may be an exception to this rule. The surge resulting from a given degree of load change will be greater for a rapid change than for a slow one, again with possible exception in the case of a throttled chamber. For a given change the surge amplitude in a simple chamber is approximately proportional to the diameter of the chamber. If the chamber is big enough, the surge becomes “dead beat” and will die away after the first half cycle. A similar dead beat condition will result if the load changes are sufficiently slow. Dead beat chamber are not usually economical.

Location of the surge chamber

The length of the pipeline between the chamber and the station is controlled by the requirements of speed regulation and of pressure regulation. Many factors are involved in regulation, but, other things being equal, an increase in the length of the pipe will increase the pressure variation on load change, while a shortening of the pipe will reduce the pressure variations.

If the surge chamber is to be formed wholly by excavation; its distance from the station depends upon the configuration of the intervening ground. If, however, the chamber is wholly or partly above ground, or if the power station is underground, it is easier to reduce the pipeline length and thus to reduce pressure variations. Practical considerations impose upon the height of steel or concrete tanks a limit dependent upon the diameter, but not usually much greater than 35 m, so that the chamber which is wholly above ground is only suited to a low head development. The excavated chamber is often preferred for reasons of economy and appearance, and because it is less subject to freezing in cold location.

The economic analysis must balance the saving in chamber costs, if the chamber is moved away from the station to keep it below ground, against the added cost of strengthening the pipeline to withstand the increased pressure variations, and must also allow, if necessary, for the cost of relief valves which can be introduced to limit the positive but not the negative pressure variations. It is of course possible to omit the chamber altogether if the pressure variations can be dealt with more cheaply by other means.

Basic Design Criteria

From the foregoing it may be seen that the surge chamber design must comply with the following conditions:

i. The surge chamber must be so located that pressure variations caused by water hammer are kept within acceptable limits
ii. The chamber must be stable, i.e. the surge resulting from small partial load changes must be naturally damped and must not under any conditions be sustained or amplified
iii. the chamber must be of such size and so proportioned that :

   - it will contain the maximum possible upsurge (unless a spillway is provided),
   - the lowest down-surge will not allow air to be drawn into the tunnel,
   - the range of surges must not be greater enough to cause undesirably heavy governor movements or difficulty in picking up load.

Loading conditions to be adopted

In the event of certain electrical or mechanical failures the entire load would be rejected instantly; this might occur with the turbines at full load and with the reservoir at any level. Full load rejection must therefore be considered in every case. It is usual to consider full load rejection under two conditions:

- With the reservoir at its maximum level, in which case the maximum upsurge level will govern the top level of the chamber,
- With the reservoir at its lowest drawdown, in which case the first down-surge level may control the bottom level of the chamber if air drawing is to be avoided.

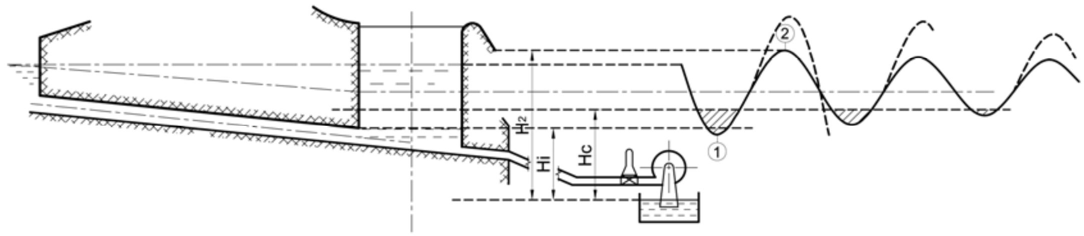
The conditions when load is “thrown on” are not quite so clear-cut as those for load rejection. When a turbine is started from rest, it first runs up to synchronous speed on no load. Under these conditions it will take about 10 percent of its full load flow. The generator is then synchronized with the system and brought steadily to the desired load. Although it is desirable that the turbine should pick up its load without undue delay, the combined operation of running up and loading is not instantaneous. If the station contains a number of manually controlled turbines, the operation of starting them all up would necessarily take an appreciable time. If, however, the turbines are automatically controlled, or if they were all running at part load, an increase to full load could take place rapidly.

An isolated station is not likely to be subjected to instantaneous large increase of load, particularly if the system load is of a diverse nature. On the other hand, if one of a number of interconnected stations is suddenly cut out as the result of a fault, the load would instantly be spread over the remaining stations in definite proportions dependent upon their governor settings. Each of these stations would then be subjected to a sudden increase of load, and the amount of increase would depend upon the total number of stations involved and the amount of load rejected by one of them.

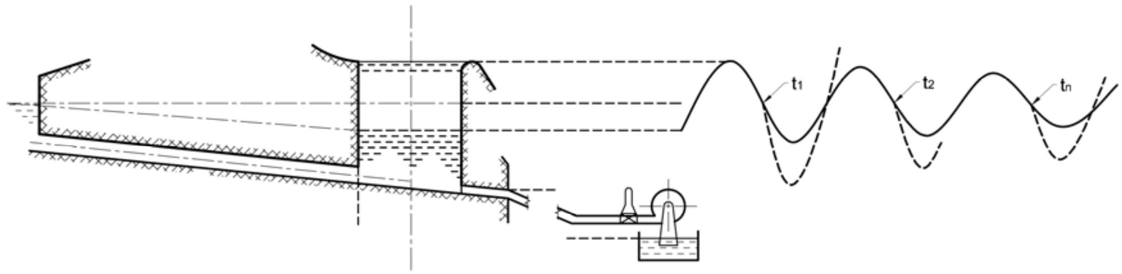
Opinion is divided upon the amount of load increase to be allowed for in design. In some cases it has been assumed that only one of the turbines is suddenly loaded , i.e. the second if there are two , the third if there are three so on, assuming that the remaining turbines are already full loaded, and relying on the contention that the operator should never have more than one turbine idling on the system. Alternatively, a percentage load increase may be adopted – a common assumption is for an increase from 75 to 100 percent. This practice is not universal and most designers adopt a load increase of 100 percent irrespective of other considerations. Whatever the load increase is adopted, it should be studied with reference to the lowest working level in the reservoir, and chamber floor should be arranged at such a level that there will be no air drawing.

The load increase conditions thus affect the conduit level at the chamber, and have also an important bearing upon the chamber design if a differential chamber is to be adopted. The load increase conditions should therefore be decided at an early stage.

Full load rejection is shown in the Figure 3.23 below. If the turbine is left “idling on the line” they will be running at synchronous speed and they will remain synchronized and ready to take load. If, therefore, they are reloaded at the instant when the water in the chamber is dropping at its maximum velocity, an extreme down surge would result, followed by an extreme upsurge as shown in the figure by broken line.



(c) EFFECT OF EXCESSIVE SURGE



(d) OVERLAPPING SURGES

Fig. 3.23: Surges under Full Load Rejection

H_c = Critical net head

H_2 = Net head between tail-water level and maximum upsurge

H_1 = Net head between tail-water level and down-surge level

By continuing the above process of loading and unloading of the turbine it would be possible to build upsurge of almost of any height. These conditions are not considered in practical design which relies on the normal and intelligent operation of the machines.

Overlapping surges have been treated by J. Calame, who suggests that they can be neglected in practice if the station contains more than one turbine. If, however, it only contains one turbine, it is desirable to consider the effect of a general short-circuit occurring a short time after throwing on full load; the condition is indicated in figure above.

The loading conditions to be adopted in final design should always be decided in collaboration with the engineers who are designing the turbines, and also with those who are to operate the installations.

Usual Design Assumptions

Whenever possible the turbine designer should provide a performance diagram showing the turbine discharge under varying conditions of load and head. Friction and other losses must be assumed. It is usual to take a low friction value when upsurge is being studied and a high one for determining down-surge levels, e.g. if the Manning's coefficient has been taken as 0.013 in calculating output and capacity, a figure of 0.012 could be used for determination of upsurge in critical area, and a figure of 0.014 for determination of down-surge.

The heads absorbed in velocity, in changes of direction of flow from conduit to chamber, and in friction losses in the chamber ports are often ignored. They may, however, be important when considering conduits in which the normal friction losses are low, or in which the chamber port consists of a long connecting shaft. Methods for the assessment of these losses are given by Calame and Gaden. Losses due to friction, and those at intakes, screens, bends, etc are usually taken as proportional to the square of the conduit velocity. Losses due to change of direction of flow at the surge chamber and port losses must be taken as proportional to the square of the chamber velocity except in the particular case of full load rejection when they may be expressed as proportional to the square of tunnel velocity. Frictional losses in the chamber itself are always ignored.

It is usual to assume that the surge results from instantaneous changes of the flow through the turbine. This is safe assumption, that the governor or relief takes an appreciable time to function. If it is necessary to allow for the rate of change of turbine discharge, the necessary data must be obtained from the turbine designer.

If the guide vane movements are assumed to be gradual, it is usual to assume also that the variation of discharge is a linear function of time, whatever may be the corresponding variation of guide vanes opening. Under special circumstances, however, it may be necessary to make a more exact analysis. In a case examined a tunnel was to be closed by a gate taking ten minutes to effect a complete closure. The movement of the gate was linear, but the discharge through it was influenced by the rapidly rising water in the gate shaft (functioning as a surge chamber) and the discharge time relation was anything but linear; neglect of those effects resulted in considerable errors in the computed surge. The assumption of instantaneous guide vane movements errs on the side of safety, i.e. the calculated surges will exceed those which will occur in practice. When computing surges for full load rejection, it is not unreasonable therefore to assume that, though the electrical load is entirely lost, the flow is only reduced by 90 percent. A similar assumption can be made when considering full load thrown on. Turbine efficiency is usually taken as constant over the range of head and load involved.

Under the steady condition, the water level in the surge tank remains constant at an elevation corresponding to the friction gradient level, and the surge tank serves no purpose.

Types of Surge Tank

Surge tanks are classified into the following types:

a) According to the location relative to the terrain,
    - Excavated surge tank
    - Free standing surge tank
b) According to the head available for the scheme
    - Surge tanks for the high head plants
    - Surge tanks for the medium head plants
c) According to the hydraulic design, they can be classified into the following groups:

- **Simple Surge Tank**

It is an unrestricted tank of constant cross section in which the maximum variation of water level is contained within the tank. This is very sluggish in action and costly since it requires greater volume. It is seldom adopted in preference to other types.

The simplest type of surge tank is a plain cylindrical shaft or tank. It is usually connected to the conduit by a short connecting shaft or port and its diameter is governed primarily by the necessity of making the area sufficient to ensure stability and, secondly, by the necessity of keeping the surges within reasonable limits of amplitude. It will be found in general that stability will determine the diameter for low heads with short conduit, while limitation of surge amplitudes will govern those with high heads and long conduits.

The simple surge chamber is a large and possibly costly work and is somewhat sluggish in action though the relatively slow movements are conducive to easy regulation. Where a chamber is excavated, due to uniformity of cross section, it will simplify and cheapen the lining.

- **Surge Tank with Sloping Chamber**

Although a simple chamber is usually vertical it is sometimes constructed as an inclined tunnel or pipe. If a circular tunnel of diameter D, is inclined at an angle of ϕ to the horizontal, its cross section area is $\pi D^2/4$, but the area of the water surface within it is increased to $1/4\pi D^2$ cosecϕ. An inclined tunnel therefore permits a given water area F, to be achieved with a smaller bore which may be safer or more expedient to drive. This type of inclined chamber has been proposed for the surge tank of Upper Tamakoshi Hydropower Project in Nepal.

- **Surge Tank with Variable Section or Expansion Chamber (Fig. 3.24)**

It is sometimes economical to vary the section of the chamber and to provide enlargements at the upper and lower levels to limit the extreme surge. This design is limited in practices to excavated chambers in which the lower enlargement takes the form of one or more tunneled galleries; the upper enlargement may either be formed in the same way or, if the topography is suitable, as a large open reservoir. The upper reservoir absorbs the rising surges, while the lower one provides reserve storage of water on starting the machines or increasing load. The upper chamber must be above maximum reservoir level and the lower one must be below the lowest steady running level in the shaft. In addition, the connecting shaft must be stable.

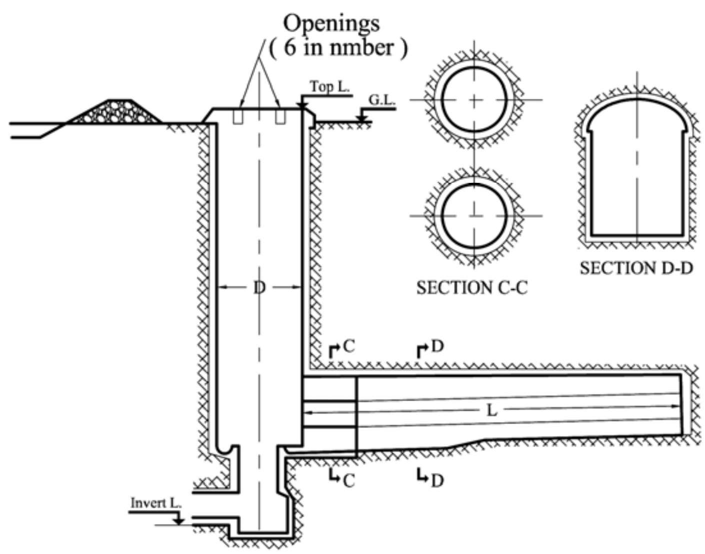


Fig. 3.24: Surge Tank with Variable Section or Expansion Chamber

- **Restricted orifice / Throttled Tank (Fig. 3.25)**

In the simple chamber, the ports between chamber and conduit have a total cross sectional area which is usually not less than the cross section of the conduit, and is sometimes greater. Under these conditions the losses in the ports themselves can usually be neglected, unless the ports are very long or the conduit very short. The surge amplitude may be greatly restricted and the chamber reduced by introduction at a throttle or constriction at the base of the chamber. The throttle increases the frictional and other losses which oppose flow between conduit and chamber. The additional friction loss is, however, proportional to the square of the velocity in the port, so that its effect is very limited except at large change of load, and the design does not lend itself readily to rapid increase of load. It is very rapid in its action; but the pressure rises are also rapid, and it is less effective than a simple chamber in relieving water hammer. At small partial load changes the throttle effect is negligible, so that the throttle is of little value in improving stability.

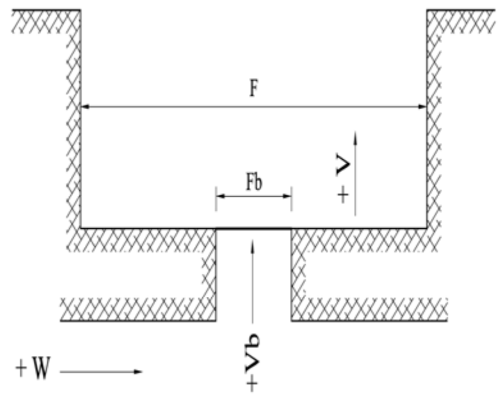


Fig. 3.25: Throttled Surge Tank

F = area of the chamber
F_b = area of the part connecting chamber and conduit
V_b = velocity at port connecting chamber and conduit
V = velocity in chamber at any instance
W = velocity in conduit at any instance

This type of surge chamber provides additional storage capacity where it is most required, i.e. at the top of the tank for load rejection and or at the bottom of the tank for load acceptance. These may be either conical tanks or tanks with expansion chambers with or without throttling and differential arrangements, and are very useful for limiting the surge heights. These are comparatively cheaper than the simple surge tanks. These types of surge tanks are used in Modi Khola, Marsyangdi and Kaligandaki hydroelectric power plants of Nepal.

- **Differential Surge Tanks (Fig. 3.26)**

The differential surge chamber is one of the earliest attempts to produce a design more economical than the simple chamber and was introduced by late R.D. Johnson. It consists of a cylindrical chamber with a central rise whose area is usually approximately equal to that of the conduit. The rise is connected to the outer chamber by ports at its base. On change of load the water level rises or falls very rapidly in the riser, thus producing a rapid deceleration or acceleration of the conduit flow, while the water level in the outer chamber moves more slowly and thus lags behind that in the riser. Though rapid in its action, the differential chamber gives reasonably low pressure rise and surge of limited amplitude.

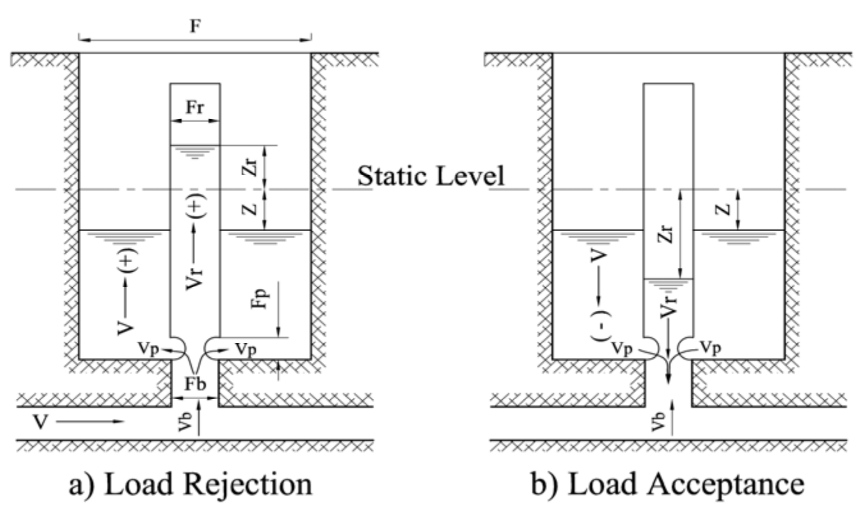


Fig. 3.26: Differential Surge Tanks

F_r = Sectional area of riser
V_r = Velocity inside the riser
F_b = sectional area of port connecting chamber and conduit
V_b = velocity at port connecting chamber and conduit
V_p = velocity in the opening
F_p = area of the opening
Z_r = water level difference between water level inside the riser and static water level
Z = water level difference between water level in the chamber and static water level

The differential chamber is a special type of throttled chamber. On load increase the differential chamber acts as a true throttled chamber, but on load rejection the spillage from the central rise prevents the pressure from reaching the intensities which occur at the base of a true throttled chamber. If the chamber is to retain its "differential" character, the riser must spill on load rejection. In order to ensure this spillage it will usually be necessary to restrict the riser ports, thus causing excessive pressure drop if full load is thrown on. Therefore the differential chamber shows to its best advantage when loading conditions permit it to be designed for partial opening as distinct from total opening. This type of chamber separates out the two functions of simple storing of additional water and effective accelerating and decelerating of water in the main conduit by the outside main tank and central riser respectively. Thus this type has got the combined advantage of simple and restricted orifice type surge tank. This characteristic of surge depends upon the area of the ports.

- **Air Cushion Surge Tank /Chamber (Fig. 3.27)**

The air cushion surge chambers represent the most recent development. With an air cushion surge chamber, the usual surge chamber vented to the atmosphere at the top of a pressure shaft or the surge tower can be omitted. The compressed air occupies from 40% to 80% of the chamber volume, and acts like a "cushion" against water hammer effects due to stops and starts or rapid changes in the power production. The air cushion surge chamber concept has been successfully used in Norway only.

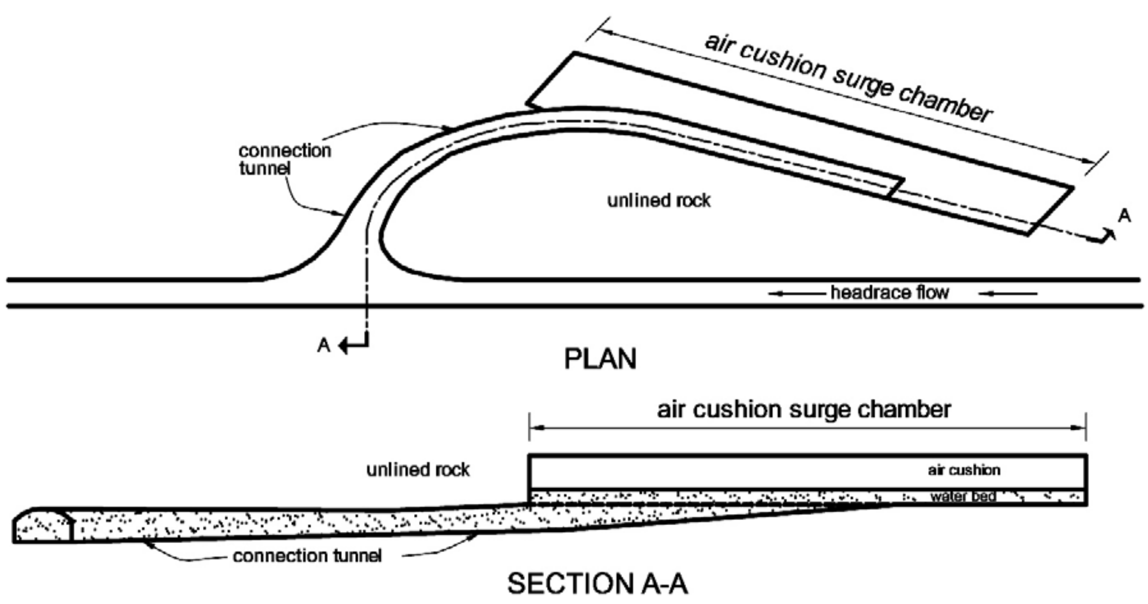


Fig. 3.27: Air Cushion Surge Chamber

The air cushion surge chamber is generally located just above the upstream part of the steel lining (penstock). When the tunnel system is filled with water, compressed air is pumped into the chamber and water bed forms the lower boundary of the air cushion.

The compressors are located in the niche downstream, the concrete plug and air is supplied via pipes through the plug. Due to dissolution in the water and leakage through joints in the rock mass there will always be some air loss. To control this air loss, cables for instrumentation are also taken through the plug.

For the design of this type of surge chambers, primarily hydraulic jacking has to be avoided, i.e. the minor principle stress has to be higher than the air/water pressure inside the chamber. Secondly, to be able to keep the air loss at an acceptable level if the rock mass permeability is not very low, it is important that the water pressure in the surrounding rock mass is higher than the cushion pressure. Hence, whenever possible, a location for the air cushion chamber should be chosen which ensures a ground water pressure higher than the planned air pressure.

In some cases, however, the hydrological conditions of the area do not make this possible, and the necessary water pressure has to be created "artificially" with a water curtain (Fig. 3.28). A water curtain is a network of boreholes which are drilled into the rock mass above the air cushion chamber and pressurized with water. Water is supplied via a pump in the compressor chamber.

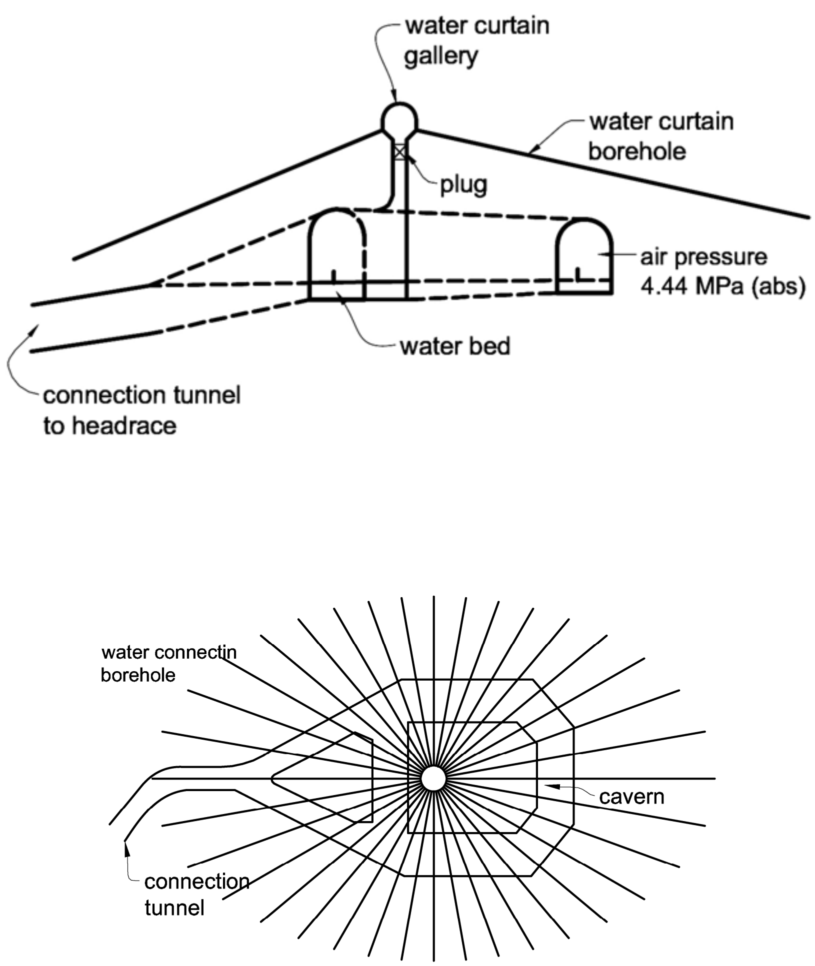


Fig. 3.28: Water Curtain

- **Surge Tank with Spilling Chamber (Fig. 3.29)**

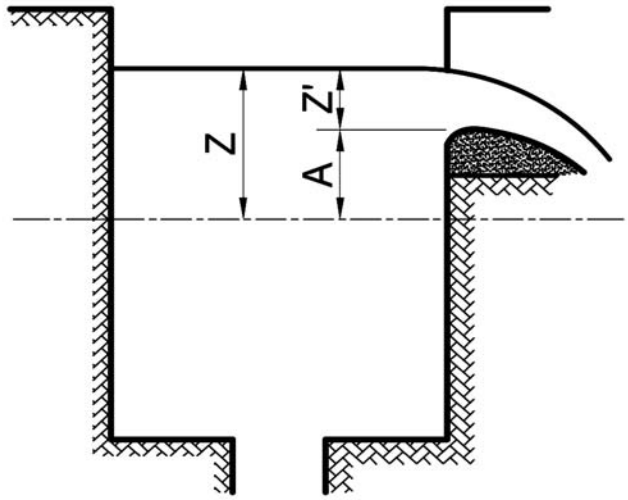


Fig. 3.29: Surge Tank with Spilling Chamber

Z = water level difference between water level in the chamber and static water level
$Z^{'}$ = water level difference between the spillway crest and water level in the chamber
A = level difference between spillway crest and static water level

Limitation of upsurge may be effected by providing the chamber with a spillway, such as that provided at Tongland, Scotland. The choice of such design is governed largely by economy and involves balancing the saving in chamber height against the cost of the spillway and the loss of water. The spillway and spillway channels must be amply designed and constructed, and are unlikely to show any great economy over the alternative of raising the chamber. If the chamber is above ground a spillway may, however, be necessary or desirable so that the depth of water in the chamber shall not be great enough to demand undesirably heavy shell thickness.

A spillway may be applied to any form of surge chamber.

- **Downstream Surge Tanks on Tailrace Tunnel (Fig. 3.30)**

Underground power stations with tunneled tailraces may need a surge chamber downstream of the turbines (possibly in addition to a chamber above them) to accommodate rapid increase of load which will back up water in the tailrace tunnel. They may have to cater for the conditions caused by the operation of relief valves which increase the momentum of the tailrace water without increasing the discharge.

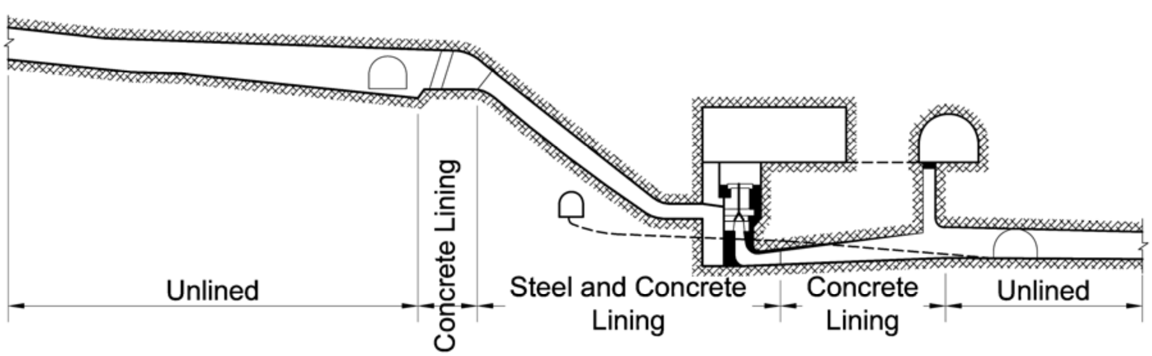


Fig. 3.30: Downstream Surge Tank on Tailrace Tunnel

Design Consideration of Surge Tank

Design of surge tank is associated with the transient analysis of the waterway and water hammer therefore, it is recommended to use a specific literature devoted to the design of the surge tank/chamber. Some of the basic surging conditions that are to be examined for the design of the surge tank are listed below.

i. Stability Condition
Small water level fluctuation during the operation should be damped and water in the surge tank should be made stable.

ii. Up-surging conditions
The top elevation of surge tank should be higher than the up-surging water level due to rapid interception of full load.

iii. Down-surge condition
The bottom elevation of surge tank should be lower than the down surging level due to rapid load increase from a half to full load rejection. In no case air should be drawn into the pipe.

iv. Damping condition
A surge tank should be damped even if the succeeding load fluctuation arises.

v. Stability of simple surge tank
Stability condition of simple surge tank is examined by Thoma-Jaeger's formula

$$h_o < \frac{H_g}{3} - \frac{H_g}{6}$$

$$F > (1 + 0.482\frac{z^*}{H_0})*\frac{Lf}{2cgH_0} \qquad \text{------------------------------------} \qquad (3.81)$$

where,

H_g = Gross head

h_o = Total head loss in headrace tunnel at maximum discharge

H_0 = $H_g - h_0$

z^* = free surging = $Q_0 / F\sqrt{Lf / gF}$

Q_0 = Maximum discharge

V_0 = Velocity in headrace tunnel at maximum discharge

L = Length

F = sectional area of surge tank

f = sectional area of headrace conduit

vi. Stability for restricted orifice surge tank
The stability is examined by Thoma-Schuller's formula

$$h_o < \frac{H_g}{3} - \frac{H_g}{6}$$

$$F > \frac{Lf}{c*(1+\eta)gH_0} - \frac{Lf}{c(1+\eta)(1+\frac{1-\eta}{1+\eta})g(H_g - Z_m)} \qquad \text{-----} \qquad (3.82)$$

$$\eta = k_0 / h_0$$

$$k_0 = \frac{1}{2g}(Q_0 / C_d F_p)^2 \qquad \text{------------------} \qquad (3.83)$$

Where,

C = Propogating velocity of pressure wave

C_d = Discharge coefficient

F_p = Sectional area of orifice

Z_m = Upper surging

η = co-efficient

vii. Up-surging , down-surging and damping

Up-surging, down-surging and damping effects are examined by the following surging equations:

Equation of motion

$$\frac{dV}{dt} = \frac{Z - C/V/V - k}{L/g} \quad \text{-------------------------------------} \quad (3.84)$$

Where, L = Length of headrace tunnel

Equation of continuity

$$\frac{dZ}{dt} = \frac{Q - fV}{F} \quad \text{-------------------------------------} \quad (3.85)$$

Where,

V = Velocity in headrace tunnel

Z = Water level in surge tank

C = h/VZ

h = Water level difference between reservoir and surge tank

k = Orifice Resistance

Method of Surge Analysis

Detailed surge analyses could be found in any hydraulic transit analysis book, and hence, they have not been dealt in these guidelines. Only the general methods used for the detail surge analysis are listed below:

i. Analytical method

- Direct mathematical solution of the equation
- Solution by using logarithmic curves
- Arithmetical step by step integration or finite difference method (Pressels, Escands methods)
- Solution by the method of characteristics
- Solution by the method of plate plane topology
- Approximate method of relative values

ii. Graphical methods

- Direction construction by Schoklitsch method
- Radial construction by Calame and Gaden method using ratios

iii. Computer Analysis

With the development of high speed, automatic analog and digital computers, the surge analysis can easily be done and correct solution obtained in a much quicker time. All the computer programs available for the surge analysis are based on the analytical methods.

iv. Model Studies

The results obtained by the above methods are often checked by conducting a physical model experiments.

3.1.1.8 Penstocks

General

Penstocks are designed to carry water to the turbines with the least possible loss of head consistent with the overall economy of the project. These are pressurized water conduits which convey water to the turbines from free water surfaces. These free water surfaces might be either surge chamber devices or reservoirs or forebays. The most economical penstock will be the one in which the annual value of the power lost in friction plus annual charges such as interest, depreciation, a maintenance will be a minimum. The variables outering the problem are: (i) daily variation of flow through penstock (ii) estimated load factor over a term of years, (iii) profile of penstock, (iv) number of penstocks, (v) material used in construction, (vi) diameter and thickness, (vii) value of power lost in friction, (viii) cost of penstock installed, (ix) cost of piers and anchors, (x) total annual charges of penstock in place, and (xi) maximum permissible velocity. It is extremely difficult to express these variables in a comprehensive formula. In addition the penstock should be structurally safe to prevent failure which would result in loss of life and property.

The penstock is usually made of steel, although reinforced, concrete, GRP, HDP penstocks have also been built recently in increasing numbers. For heads up to 100 meters even wood stave penstocks have been applied.

The strength and flexibility of steel make it best suited for the range of pressure fluctuations met in the turbine operation. Present design standards and construction practices were developed gradually, following the advent of welded construction, and are the result of improvements in the manufacture of welding-quality steels, in welding processes and procedures, and in inspection and testing of welds. The design considerations of penstock pipes in general depend on the following:

Owner's Requirement

The owner requirements must consider the following:

- Preferred material and design type
- Plant operation requirement
- Annual cost of capital investment and cost of power and revenue loss
- Inspection and maintenance provisions
- Applicable internal and governmental guidelines, criteria, and design requirements
- Legal and political issues, including environmental and licensing issues.

Site-Specific Requirements

The site specific requirements are also equally important due to environmental restraints, limitations on size and weight, geological restraints, hydrologic considerations, and limitations (alignment and support) to the penstock physical layout. The choice of type of installations such as above ground or buried penstocks or tunnel liner will be dictated by site specific conditions, requirements and design and cost associated with such layout. Therefore the following points must be considered for site-specific requirements to be addressed:

a. Land ownership, right-of-way limitations, mineral rights, and limitations relating to excavation/quarrying operations

b. Environmental restraints, including aesthetics, fish, game and wildlife preservation, archaeological excavations, disposal of material, clearing and erosion

c. Terrain configuration

d. Site geology, hydrology (groundwater conditions) and soils

e. Applicable codes and mandatory requirements

f. Other site-specific considerations

Preliminary Study

The preliminary study phase is an important phase of the general design effort of the experienced designer. The final penstock configuration, alignment, design, and requirements and parameters must be determined during this study phase. The designer must investigate the site conditions and make several layouts of various alignments. Terrain, geologic characteristics, and foundation conditions play important roles during this study phase. Since the ultimate goal of this study phase is to determine the most economic and implementable alignment, it is not necessary to approach the study in a great precision.

Type of installation

The type of installation selected should reflect the above consideration. Penstocks are classified into different types depending on their general features. Three following types have been designed and used in recent years:

Supported penstocks (Exposed Penstock): These are usually fabricated from steel, plastic fibreglass or wood stave pipe. They can be located above the ground or in a none encased tunnel and are usually supported on either steel or concrete support systems. Plastic or fibreglass penstocks should not be exposed to sunlight because ultraviolet rays break down the material.

Buried penstocks (Underground): These are usually fabricated from steel, concrete, plastic or fibreglass. They can be either partially or fully buried.

Steel Tunnel Liner (Underground): These are located in a tunnel and fully encased in concrete or encased in a portion of a dam. The type of installation selected should reflect the cost-effective penstock system which should consider the technical, environmental, economic and constructability factors.

Each penstock type has different associated design, material and construction costs.

Scopes

The scope of this guideline related to the penstock covers the following:

1. Preliminary determination of diameter
2. Economic diameter determination
3. General design criteria
4. Different types of loading condition
5. The special case covers the specific guideline for each type of penstock.

Conditions governing the Adoption of a Pipeline

When a hydro-electric power station is to be supplied with water through a tunnel, arrangement most frequently adopted is to terminate the tunnel with the portal at a relatively high level on the hillside and connect the portal to the power station by means of a steel pipeline. The alternative to such a pipeline is to arrange the tunnel at a low level so that it connects direct to the turbine. Whether a pipeline or a low-level tunnel is adopted is largely governed by the overall costs, and many factors have to be taken into consideration, including the nature of the ground, whether composed of sound rock or deposited material; the amount of rock cover above the tunnel; the most desirable position of the portal having regard to the most suitable geology.

Projects with penstock started from the forebay in Nepal are:	**Projects with penstocks started from the surge tanks in Nepal are:**
- Trishuli HPS - Devighat HPS - Sunkoshi HPS - Puwa Khola HPS and others	- Kaligandaki HPS - Marsyangdi HPS - Kulekhani I, II HPSs - Khimti HPS - Bhotekoshi HPS - Chilime HPS - Modi Khola HPS and others

Having decided to adopt a pipeline and having provisionally located the centreline in plan and elevation, it is necessary to determine the water pressures to which it will be subjected. These pressures are plotted on the Design Pressure Diagram as shown in Figure 3.31 below, and include the static pressure due to the level of water in the reservoir, plus and increase on account of surge and water hammer to give the design head.

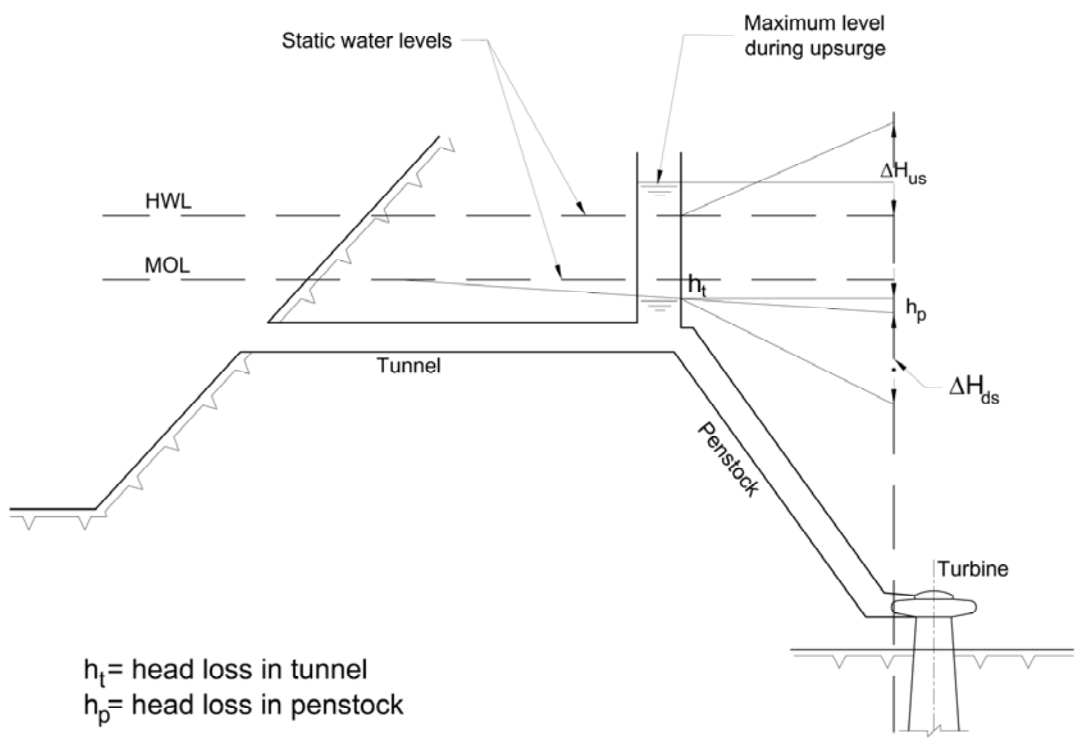


Figure 3.31: General Water Conduit Profile with Design Pressure Diagram (Source: Handbook on Hydraulic Computations by P.G. Kiselev, 1957)

It can be assumed that the maximum pressure through the pipeline due to surge is equal to the head resulting from the maximum level of water in the surge shaft. It should be noted that the water-hammer pressure wave travels from the valve to the surge shaft in a few seconds. The pressure is maximum at the turbine valve and decreases to zero at the free water surface in the surge shaft / tank.

As the water-hammer pressure wave travels rapidly, the amplitude of the successive pressure waves quickly falls from a maximum to a negligible amount, and normally this latter stage is reached before the surge pressure attains its maximum value.

A water-hammer pressure oscillates above and below the immediately previously existing water pressure line, the pipeline must be investigated to ensure that the water pressure does not fall below atmospheric pressure. This undesirable condition is most likely to occur where there is a sharp bend downwards, in the pipeline. The pressure occurring at this point should be determined for the conditions when the reservoir is at its lowest working level and a turbine has been started up thereby producing a downsurge in the surge shaft. It will generally be found that the negative water hammer due to opening of the turbine valve is greater than the amplitude of the secondary water hammer oscillations due to closure.

Alignment

To determine the most economical alignment of a pipeline, the designer must investigate the site and make various layouts on topographic maps. He must then estimate material quantities for each layout and evaluate its constructability.

When making these layouts, the penstock should be located on stable foundation sites such as along a ridge or a bench that has been cut into the mountainside, Avoiding of troublesome sites such as underground water courses, landfill, fault zones and potential slide areas is quite important.

Because low-head penstocks cost less than high-head penstock, the pipeline at high elevations needs to be made as long as possible before going down the mountainside into powerhouse.

To minimize costly anchors and costly pipe transition sections, vertical bends, horizontal bends, and changes in diameter should be combined in a way to have them at the same location.

Economic Diameter

The economically justified diameter for a penstock required to carry a design flow is the one at which annual cost due to the greater investment do not exceed the annual value of the resulting incremental energy output. The governing criterion is thus to regain economically the last incremental kilo-watt-hour made available by reducing the head-loss through using a larger diameter.

Under average conditions and present day prices the most suitable diameter for pipeline is frequently one which gives a maximum water velocity of approximately 5 m/s (15 ft. per sec.). As the pressure head or the cost of the pipeline per ton increases, it is economical to reduce the diameter of the pipeline; similarly, as the load factor on the station increases, the diameter should be increased. For a particular station the only variable is the pressure head which increases as the pipe runs downhill towards the station. Some pipelines are therefore reduced in diameter at the lower levels.

In the early stages of a project it is necessary to determine the approximate diameter of a pipeline for estimating costs or other purposes.

In schemes where there are a number of turbines a single pipeline to supply all the turbines will involve the use of the least amount of steel. Making joints in thick plates and handling large pipes, however, are difficult and costly. It is frequently desirable, therefore, to use a separate pipeline for each turbine, at least for the part nearest the power station where the pressure is greatest. When separate pipes are used portal valves can be introduced on each pipeline, thereby enabling any branch pipeline to isolate for maintenance without affecting the operation of the remaining turbines.

In order to determine the economic diameter for any particular pipeline with greater accuracy, it is necessary to consider a number of alternative diameters for the pipeline, and then estimate for each diameter the annual charges to cover the capital cost of construction and the annual value of the electrical energy lost on account of the friction head in the pipeline. The diameter which gives a minimum for the sum of these two quantities is the most economical diameter.

Determination of Economic Diameter or Optimizing Penstock Diameter

In additional to alignment and design head, it is important to know about plant operation and other factors that determine the annual cost of constructing and operating a powerhouse penstock. The two major cost items involved in the annual cost are (1) cost of capital investment and (2) cost of energy revenue loss from frictional head loss.

- Cost of capital investment (C_t): The initial investment (capital) cost must be paid off over a period of years (project life) at a specified interest rate. When the project life and interest rate known, the capital recovery factor (CRF) can be determined. By multiplying the capital cost by the CRF, the annual cost of capital investment is calculated.
- Cost of energy revenue loss (C_e): The flow rate (Q), the cost of kilowatt hours generated each year, and turbine-generator efficiency must be determined through careful study and planning. In addition, head loss must be accurately determined. When all these parameters are known, the annual cost of energy revenue loss can be calculated.

The total annual cost is determined by adding the two major costs above. Finally, select a diameter that minimizes the total annual cost. The shell thickness is usually governed by the allowable stress.

It can be shown that the flow rate (Q) is by far the most important parameter and must be selected carefully because of its impact on the total annual cost and penstock size. The figure presented below (Fig. No. 3.32) will show the relation between cost and optimum diameter.

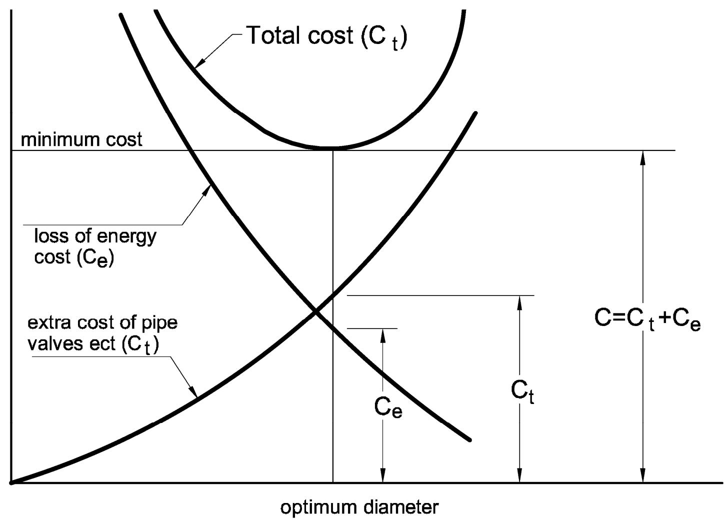


Figure 3.32: Penstock's Optimum Diameter

The economic diameter of a penstock is a function of head loss, cost and the value of energy. A first estimate can be obtained from the equation 3.86 given below:

$$D = E_P \times P^{0.43} \times H^{-0.57} \qquad (3.86)$$

Where,

E_p = 0.49
D = diameter in meter
P = turbine rated capacity (kW)
H = turbine rated head (m)

Economic Diameter Equations

The economic diameter equations for penstocks are developed for the Case 1 or Case 2. They are as follows.

(1) Case 1 —Minimum thickness for shipping and handling

When the shell thickness (t) in millimetres is determined by D/288, the economic diameter (D) is given as:

$$D = 0.9025\left[\frac{fhMEQ^3(pwf)}{WC}\right]^{0.1429} \quad \text{-------------------------------------} \quad (3.87)$$

When a specific value is used for t, the economic diameter is:

$$D = 0.3867\left[\frac{fhmEQ^3(pwf)}{WCt}\right]^{0.1667} \quad \text{-------------------------------------} \quad (3.88)$$

(2) Case 2 –Internal pressure governs

The economic diameter (D) for a Case 2 penstock is given as:

$$D = 0.5\left[\frac{ShfMEQ^3(pwf)}{WCH}\right]^{0.1429} \quad \text{-------------------------------------} \quad (3.89)$$

Where,

f = friction factor

h = hours per year of operation

M = \$/kWh, composite value of energy

E = turbine/generator efficiency in decimal form

i = interest rate

n = years, repayment period

Q = design flow

W = specific weight of steel

C = capital cost of penstock installed

pwf = present worth factor,$\{(i+1)^n+1\}/i(i+1)^n$

S = allowable stress

t = thickness, D/288

H = Design Head

It is very important to know, how the power plant will be operated when determining the penstock diameter. Some of the parameter values, such as the competitive value of power and the number of hours of power generation, will vary greatly for a base-load power plant compared to a peaking load plant. Selecting the design flow (Q) is important because this term is cubed in the diameter equations. Variations in the design flow value have a significant impact on the resulting diameter calculations.

Water Hammer

Water hammer is the result of a change in flow velocity in a closed conduit causing elastic waves to travel upstream and downstream from the point of origin. The elastic waves, in turn, cause increase or decrease in pressure as they travel along the line, and these pressure changes are variously referred to as water hammer, surge or transient pressure.

Basic Relationships:
The following fundamental relationships in water hammer or surge-wave theory determine the magnitude of the pressure rise and its distribution along a conduit. The pressure rise for instantaneous closure is directly proportional to the fluid velocity at cut-off and to the magnitude of the surge wave velocity.

1. Mean velocity of flow in penstock

$$V_o=Q/A_o \quad \text{-------------------------------------} \quad (3.90)$$

$$A_o=L/\Sigma(L_i/A_i) \quad \text{-------------------------------------} \quad (3.91)$$

Where,

V_o = Mean Velocity of flow
A_o = Average Sectional Area
A_i = Cross-Sectional Area of i^{th} section
L = Total length of the penstock pipe
L_i = Length of the i^{th} section

2. Velocity of Pressure Wave

$$a=1/[\sqrt{w/g}\{1/k+1/E*D/t\}] \quad \text{-------------------------------------} \quad (3.92)$$

Where,

a=Velocity of water hammer wave (m/s)
D=Average pipe Diameter.(m)
t=Pipe Thickness (m)
w=unit weight of water
g=Acceleration due to gravity
k=Elasticity Modulus of water
E = Elasticity Modulus of steel

The parameters mentioned below are quite important for determining water hammer pressure in penstock by using Allievi's diagram. The Allievi's diagram for water hammer (Ref. Fig. 3.33) is the relation between three parameters namely ξ , θ , and ρ represented by the following relations:

$$\theta = T/(2L/a) \quad \text{-------------------------------------} \quad (3.93)$$

$$\rho = aV_o/2gh_o \quad \text{-------------------------------------} \quad (3.94)$$

$$\xi = (h_o+h_m)/h_o \quad \text{-------------------------------------} \quad (3.95)$$

where,
h_0 = Static Head,
In which, hm stands for rise in pressure due to water hammer.

By calculating θ and ρ, the value of ξ can be found from the Figure 3.33 and then using the relation ξ, static head we can easily determine the water hammer head from above relations. Accurate results of a water hammer analysis depend on knowing the various hydraulic and physical characteristics of the system. The velocity of the pressure wave is a fundamental factor in any water hammer study, as the water hammer pressures are directly proportional to its value. This velocity depends on the pipe diameter, wall thickness, material of the pipe shell, and density and compressibility of the fluid in the pipe.

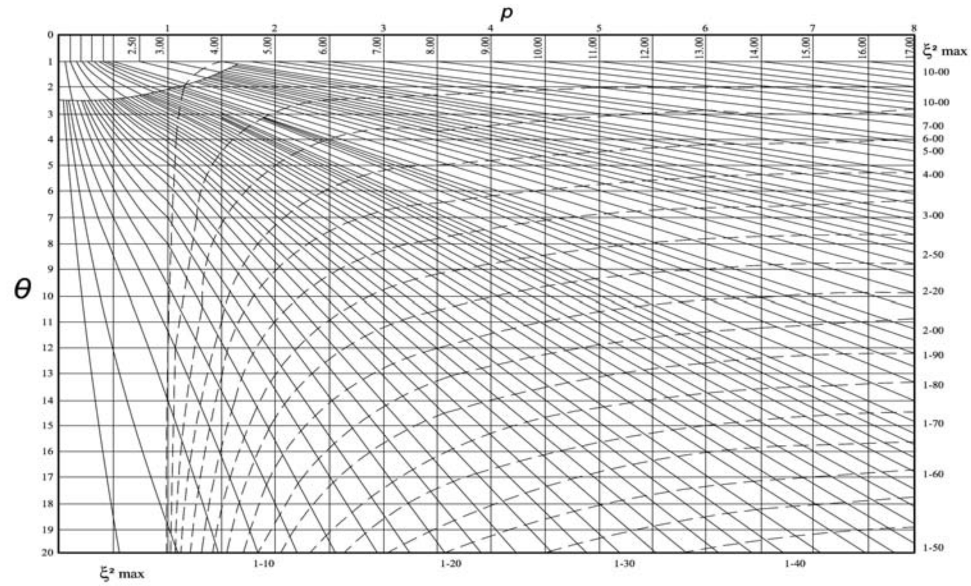


Figure 3.33: Allievi's Diagram for Water Hammer Calculation*

Calculation of Head Losses

Hydraulic losses in a penstock reduce the effective head in proportion to the length of the penstock and approximately as the square of the water velocity. Accurate determination of losses is not possible, but estimates can be made on the basis of data obtained from pipe flow tests in laboratories and full-scale installations. The various head losses which occur in penstock are the friction, bend, gradual contraction, bifurcation, valve and others.

3.1.1.9 Tailraces and Outfall Structures

Tailraces

The tailraces generally convey water from the powerhouse after use in power generation to the downstream channel. In case if the tailrace water is, again, used in the second power station, the tailrace directly conveys water to the second power station like in Trishuli-Devighat cascade and Kulekhani-I and II cascade. The tailraces could be open canal, closed conduit or tunnel depending upon site situation and layout design. As with the headrace canal the tailrace enables maximum head utilization at a site. Often headrace canal (for generalization say headrace conduit) and tailrace are used together to develop the full head. The basic design principles presented for waterways (section-3.1.1.3) apply to tailrace design as well.

Outfall Structures

The outfall structures particularly of hydropower projects are generally located on river / stream banks. They are required for protecting the natural terrain from erosion. Some of the outfall structures particularly the outfall structures of tailraces normally continuously operated, while the others such as outfall structures of overflow spillways constructed at approach canal and forebay will not be under frequent operation. Similarly, the outfall structure of silt flushing pipes constructed at the desander will be operated mainly during monsoon when the silt carrying capacity of the rivers is the highest. The outfall structures constructed at the upper hydropower plant (refer Trishuli) of a cascade will operate only for few hours / days when there will be shut down in the lower hydropower plant.

The design aspect more concerned with these structures deals with the defined flow diverted from the river. Hence, the energy dissipating structures at the outfall structures are comparatively of smaller magnitude than those required at downstream reaches at dam / headworks. For design guide of energy dissipating structures refer sub-section: “Energy Dissipaters” above.

* (Source: Water Hammer by Dr. Charles Jaeger, Hydroelectric Engineering Practice, Volume I, Civil Engineering, Second Edition, CBS Publishers & Distributors, Edited by Brown J. Guthrie)

3.2 Structural Design

3.2.1 Structural Design - General

3.2.1.1 Introduction

This chapter focuses on the process of design, guides through steps of design and refers to other sources to provide detailed information. As with the designs in general, assumptions are to be stated early and clearly in the process. This design-guideline is based on the assumption that the designer has prepared the general arrangement and identified the essential facilities.

3.2.1.2 Design Process Overview

Collection and Review of Available Information

Many elements are needed to come in the structural design from different and often incompatible sources to create a single whole. For example, hydrology provides loading data and gives flood and overflow information, while hydraulics set the constraints that influence layout and sizing. Geotechnical information is important in determining allowable foundation pressure and in siting and layout. Also it determines the seismic characteristics of the site, defines on the location and characteristics of faults, and the specific geology of the site and region.

At present more than a hundred sites hydroelectric developments have been already studied in Nepal, but the level of study varies. Reports and studies available with different agencies contain a wealth of information that can be used by the structural designer(s) of the conveyance system. The preliminary design process can be greatly simplified if someone else has completed the process in the previous study.

Preparation of Design Criteria Memorandum

The design criteria memorandum is a document which describes physical data, testing and studies, methods of determining loading, assumptions used, and analysis and references used in the design. There are many existing design criteria available for the design of hydroelectric structures. If the design falls under the jurisdiction of a governmental agency, the designer is encouraged to verify the design criteria with that agency. The US Army corps of engineers, the US Bureau of Reclamation, the Federal Energy Regulatory Commission, and state and local agencies typically have existing design criteria or have regulations which must be considered in developing the criteria. In Nepal, design based memorandum (DBM) is prepared for individual project and used for the detailed design of the project. DBM prepared for Kaligandaki “A” hydropower project could serve a ready reference.

Determination of Loading

Loads are generally based on:

- Climate – ice and snow loads
- Geographic location – seismic and geotechnical loads
- Hydrology and hydraulic – normal and high water levels, wave and surge forces, hydrostatic forces, and uplift pressures
- Gate loads and transients
- Geotechnical soil and silt loads
- Structural systems

Preliminary Layout and Sizing of Structures

Coordinate with other disciplines to determine what the anticipated requirements will be. The final information may not be available, but best guess-estimates and assumptions allow the designer(s) to continue on a preliminary basis.

Hydraulic Considerations:
The primary function of the conveyance structure is to deliver required quantity of water to the power house for energy production. Hydraulic considerations make up a large portion of the criteria which contribute for the design. A good design should have:

- No vortices
- Smooth flow over the whole conveyance system
- Minimum flow separation
- Minimum head losses

Structural and Geotechnical Considerations:
Structural and geotechnical engineers can greatly influence the layout and sizing of structures with their early involvement. Their role is often that of making requirements of other disciplines work together as a coherent whole.

Geotechnical considerations can be put into two categories: (a) load producing, and (b) load resisting. Load producing considerations include: silt load, active or at rest earth pressure, and rock loads. Load resisting considerations can be further subdivided into positive characteristics and limiting items. Positive characteristics include soil bearing capacity, frictional resistance, and passive earth pressure. Limiting items reduce foundation load capacity or make it unsuitable in other ways, such as deformation characteristics, creep potential, and deep-seated failure planes.

Construction Considerations:
Constructability (usually the structural engineer's responsibility) must be considered during the preliminary layout and structural design stage.

Mechanical Equipment:
The design and layout should accommodate the control equipment typically found in the conveyance system. That equipment may include gate controls, batteries, water level meters, ventilation, etc.

Maintenance Considerations:
The conveyance system layout should consider safe and easy access to all equipment. Control panel doors should be located so that the door swing does not create a safety hazard. Clear and unobstructed walkways should be identified early in the design as this "unused" space will greatly affect the size and orientation.

Preliminary Stability Analysis

Before too much time and effort are spent on a design, stability should be checked to determine whether additional measures are needed to achieve adequate stability. These measures include rock anchors or foundation drains, which can have a significant effect on project costs and scheduling of time required for design and preparation of documents. The preliminary base pressure and concrete stress should also be checked at this time. Planning for the further final analysis is also useful. Stability analysis lends itself to automation in some degree by use of spreadsheets alone or in combination with other mathematical computer applications. By taking the time during the preliminary design to streamline the analysis process, the final design can become a much simpler task.

Layout

The layout proceeds concurrently with conveyance siting and the preliminary stability analysis phases of the design. Once preliminary stability has been completed and the workability of the conceptualized conveyance system is known, then the layout can be finalized, features identified, and the design can proceed ahead. The designer should categorize the assumptions which serves as a basis for the layout. As the design evolves these assumptions should be confirmed.

System Analysis

System analysis describes analysis that takes the individual conveyance system as a single unit. This consists of stability analysis and resultant stresses in the concrete and foundation. This type of analysis is considered mostly for the exposed conveyance system like canal, desanding basin, forebay, aqueduct, etc. but does not apply to the underground conveyance system like tunnel.

Stability:
Loading cases and factors of safety used for stability will depend on whether or not the structures are a water retention structure and the applicable requirements for stability are set.

Foundation Pressure and Frictional Resistance:
Allowable foundation pressure and frictional resistance are usually set by a geotechnical engineer. The foundation pressure, which results from the stability analysis, must be within the allowable limits. Structure-ground interaction is a critical aspect of the design. If a zero settlement criteria is established, then extensive foundation analysis and preparation may be required. However, the structure could be designed to accommodate a determined amount of settlement by creatively using expansion and contraction joints.

Concrete Stress:
The most common approach is to use the two-dimensional gravity method or limit equilibrium method to determine the stresses in the concrete and foundation simultaneously. Finite element modeling provides highly accurate results as it can more accurately simulate the effects of geometry. Concrete stress determined by a simple analysis method may not provide an accurate representation, especially for desanding basin and forebay with complicated geometry.

Component Analysis

Component analysis considers individual items that make up a conveyance system, such as gates, hoists, gate slots, trashracks, air vents, concrete walls and slabs roof system, valves, and bypass piping.

Detailed Design

During detailed design, structural members are sized and reinforcing steel requirements are calculated. Structural construction joints and water stops are located and added to the drawings. Specifications are prepared, describing materials and methods.

Drawings and Specifications

Drawings and specifications are the main tools a designer uses to direct the contractor in the construction of the structures in the project. As with any form of communication, success depends on clear understandings between parties. Organizing the specifications using a conventional method like the Construction Specification Institute's format or government specification will improve the reader's understanding.

The following list is taken from a variety of projects, and is intended to provide guidance as to the breakdown and type of features presented on drawings.

- Location and layout plan
- General plan
- Individual structure’s plan and location

- Individual structure sections
- Reinforcing details
- Trashrack
- Trashrack frame
- Gates
- Miscellaneous metal
- Mechanical equipment and details
- Electrical equipment and arrangement
- Instrumentation and control

Coordination:
The drawings and specifications for the conveyance system must be integrated with other principal project features.

Design Changes:
Design changes should be documented and approved to ensure that changes are recorded consistent with design intent.

Construction Completion Drawings:
After the project is completed drawings are prepared to provide a record of the structure as it was constructed. These are referred to as "record drawings” (formerly referred to as "as built" drawings). These drawings include actual foundation levels and any changes in the design or construction from the bid or released for construction drawings.

Quality Control and Assurance

A program of quality control and assurance reduces risks to public and employee safety, reduces professional liability, and reduces possibility of costly wasted time and revisions due to calculation or drawing errors.

Check Calculations

Checker should evaluate:

- Codes, standards, and regulatory requirements are correctly selected, referred to and applied
- Assumptions are based on sound engineering principles and are all adequately documented and ultimately confirmed.
- Applicable construction and operating experiences are considered
- Appropriate calculation methods are used
- Load combinations are reasonable and foreseeable
- Conclusions are reasonable compared to assumptions
- Mathematical accuracy and safety margins
- Appropriateness of any computer programs used
- Correctness of computer input and reasonableness of output

Check and Revise Drawings

Checker should evaluate:

- Completeness: Whether the drawing contains or refers to everything that the contractor will need?
- Compatibility: Are the objects shown on different views compatible with the other views of the same object? Other portions of the project? Do they properly reflect the design calculations?
- Clarity: Does the object shown convey the information in a way that the contractor can understand easily? Are cross references to applicable drawings and sections of specifications clear and up to date?

Design Review:
The review should be performed by people familiar, but not directly associated with this portion of the project. They may identify conflicts in the design like the reinforcing steel congestion at a column-girder-slab connection that will not allow the concrete's maximum size aggregate to pass through the cage and so on.

Constructability Review:
The most elegant design is useless if it can not be constructed economically. The construction process is examined step by step considering access, equipment operation, seasonal restriction, permitting, scheduling interferences, ease of water, safety, and economics. It is often useful to have someone experienced with construction on similar projects, evaluate constructability. Total construction time may be evaluated for successful completion of the project's cost and schedule objectives.

3.2.1.3 Design Criteria

Purpose

Usually a memorandum is prepared to provide the basis for design of all project elements. The design based memorandum (DBM) is a checklist of considerations and references used in preparation of detailed design drawings, calculations, and specifications for civil and structural aspects of the hydropower components at large including conveyance system for this design guidelines. Layouts and designs developed using design criteria are also used to prepare project construction cost estimates. Well prepared design criteria should provide information or references required for an independent check of project design.

Contents

The scope depends on this size and nature of a project, and its organization reflects both the project and the preparer. Conveyance system DBM will be one of the documents along with the other DBM documents prepared for the whole project components, and it is important that project documents are uniform. The following items should be considered, but order and presentation could vary depending on the project.

Introduction:
An introduction presents the project objective, purpose, and scope of the design criteria, states level of details and portions of the project covered.

References:
A list of applicable references allows an easy overview of available resources. Appropriate items include: design criteria for other portions of the project, relevant sketches and drawings, reports, articles, reference papers, codes, and standards.

Description:
The design criteria should include a brief general description of project location, layout, general dimensions, project elements, and construction sequence.

Operational Criteria and Serviceability:
The conveyance system particularly the desanding basin and the forebay operation is an important consideration in design. It is important to document how the designer has envisioned the operation of individual components of the system.

While preparing the criteria, the designer should plan for future maintenance of the structures as well. The designer should also provide hatches and openings to make it possible to remove items that may be replaced in the future.

Determination of Critical Items:
The critical items are structures, systems, or components determined to be essential for safety, compliance with agency regulation, or efficient facility operation.

The following criteria can be used for identifying critical items. These may provide useful insight into the decision making process when choosing critical items.

- Failure or malfunctioning would have a significant adverse impact on personnel safety, safety and welfare of the general public, the environment, efficiency of operation, or unplanned shut-down of an operating unit, operating license or other regulatory requirements.
- Replacement or repair activities could result in a significant cost increase, or excessive delays in project or outage schedules.

Design Requirements and Methodology:
- **Hydraulics and Hydrology:** Design criteria lay the groundwork for determination of water-related loads on the project, and also provide the basis for dimensioning of elements such as trashracks, forebay, transition to pipelines, gates, stoplogs, and air vent size. The hydraulics and hydrology sections describe the background for determining the physical data used in the design. These include studies, methods, references, and requirements used to determine the physical data listed below.
- **Layout and Depth:** Requirements and methods used to determine size of the canal, pipe, tunnel, desanding basin, forebay, penstock pipe submergence, and anti-vortex devices to prevent flow separation, etc.

Physical Data:
Among the items considered as physical data are the following:

- Maximum and minimum water levels associated with loading cases
- Range of flows
- Allowable velocity of water through canal, trashracks, desanding basin and the forebay
- Flow distribution
- Required submergence
- Temperature range
- Air velocity through vent
- Ice characteristics
- Seismic acceleration

Design Loads:
The design loads may be given either as actual loads or as methods by which the loads are calculated. Some loads that should be considered are as follows:

- Dead load
- Live load

- Soil Load
- Hydrostatic Load
- Hydrodynamic Load
- Seismic Load
- Ice Load if applicable
- Equipment Load

Materials:
The materials section addresses:

- Concrete mix
- Reinforcing steel grade
- Trashrack steel grade
- Unit weights of concrete, soil, silt, water, bedrock
- Special requirements for materials (rebar near water, air entrained concrete)
- Waterstops and seals
- Material properties (i.e., compressive strength, modulus of elasticity)
- Allowable bearing strength of the foundation
- Shear resistance of the foundation

Structural Analysis:
The following items should be listed:

- Load combinations and type of analysis
- The method and criteria used for establishing sliding stability and floatation of the structure
- Allowable stresses and load combinations
- Methods of analysis (i.e., elastic, finite element), codes and standards which establish acceptable methods
- Factors of safety

3.2.1.4 Design Loads

Dead Loads

Dead loads mainly consist of structural weight, superimposed backfill, and weight of permanent equipment such as gates. Often preliminary design begins before actual structural systems are determined and material weights are known. In the absence of actual test data, the unit weight of structural concrete can be taken as 2.4 tons per cubic meter. Final design should consider actual material weights determined by laboratory testing, if available.

Geotechnical Loads

Soil Loads:
Soil and silt loads that are described in design criteria should represent conditions that will be anticipated for the effective life of the project. Sedimentation studies can establish desanding basin and forebay silt or soil levels.

Soil loads can be modeled as active, passive, or at rest pressures sometimes presented as equivalent fluid pressure analyzed as a fluid load. The type of soil pressure used will depend on the application. Often the decision will be whether to use active soil pressure or at rest soil pressure for structural or stability calculations. In general, active soil pressure should be used against surfaces that will flex under soil loads, and at rest pressure against rigid surfaces. The amount of displacement/rotation laterally away from soil being required to produce active earth pressure conditions is quite small, and a flexible structure is always very likely to deform sufficiently for the active pressure to be developed. A very rigid structure might shear along the base without active pressure being allowed to develop.

Passive pressure is mobilized by movement of the structure into a soil or rock mass. A typical application of passive soil pressure is in the design of anchor plates or blocks embedded in soil with a tension rod or cable oriented so that cables pulls a block against the soil. Engineering judgment is required to determine what is the most appropriate pressure for the application.

Saturated Soil Loads:
Saturated soil loading above the prelatic surface, the weight of water is subtracted from the saturated weight of the soil, and the soil loading coefficient is applied to the buoyant weight alone. The water load is applied without reduction factors.

Surcharge Loads and sloped Surfaced Embankments:
Renkine's theory addresses surcharged or sloped embankment loadings on structures.

Silt Loads:
Silt loads can be modeled either as a saturated soil or as an equivalent fluid. The US Bureau of Reclamation (USBR) gives a commonly used equivalent fluid load in Design of small Dams (USBR, 1974) to be used in the absence of test data.

Rock Loads:
Rock loads are not generally considered as a design load on the structure itself. In most cases, the potentially loose rock mass is stabilized with a separate, self contained system. Conditions that could require that protective measures be taken include highly weathered rock formations, fractures within the rockmass that could result in a bock of rock becoming detached from the parent block, or undercut rock formations.

Potential solutions for unstable rock formations are rock bolts, reinforced shotcrete, wire mesh cover over the rock face, pre-stressed anchors, and pressure grouted soil anchors. The type of stabilization will depend on the permanence of the solution and the size and type of rock mass to be stabilized. References for these techniques include: Post Tensioning Manual, 1990; Foundation Treatment (Swiger, 1988); and rock Reinforcement Engineering Manual.

Typically the designer presents the existing topography and the excavation drawing in the contract documents. Rock stabilization required for safety during construction is usually the responsibility of the contractor. Because the designer has access to and knowledge of the site, the contractor may expect or otherwise ask for guidance from the geotechnical or structural engineer on methods to support a temporary cut slope.

Seismic Considerations:
At sites where seismic activity is anticipated, there is potential for soil structure interaction and liquefaction.

Hydrostatic and Hydrodynamic Loads

Hydrostatic Loads:
Hydrostatic loading consists of the horizontal loads from the outer surface and inner surface water pressure. The water pressure is taken to be the weight of the water times its height.

a) Outer surface load: The outer surface hydrostatic pressure is the weight of the water times its height. This results in a triangular distribution of load on a vertical face. The hydrostatic pressure should continue below the soil level to the base of the structure.

b) Inner surface load. Water flowing inside the conveyance system will have the same force as it is in the outer surface walls depending on the height of the water inside the conveyance system.

c) Uplift, Rock foundations. Normal uplift on rock foundations without cutoff or drainage is often assumed to be the linear gradient from full headwater at the upstream end to full tail-water (if appropriate) at the downstream end.

d) Uplift, Soil Foundations. The uplift characteristic of a structure on a silt foundation can be found by performing a seepage analysis of the structure for each of the different loading conditions. Uplift will be equal water pressure determined by the results of this analysis. Two acceptable methods for determining the magnitude of uplift pressure are : the Creep Theory and the Flow Net Method. The creep method calculates the distance that a molecule of water would have to travel as it flows beneath a structure. The flow net method divides the flow under the structure into a number of channels of equal flow and into equal potential lines. The flow and potential are used to calculate the uplift pressures.

e) Other Water Loads to be Considered: The weight of water within or supported by the structure is included in the analysis as appropriate. For example, the normal case structural weight calculated for the stability calculation should include the weight of water within the structure.

- Weight of water in structure will depend on the load case; water may be treated as dead load for certain cases of stability analysis.
- Weight of water on lip or sill

Hydrodynamic Loads:
The hydrodynamic loads are caused by accelerating water, wave, current forces and hydraulic transients, or water hammer effects.

Seismic Loads

Structural Mass:
The seismic case considers the mass of the structure to be analyzed accelerated at the level of the design earthquake. Other related design components are base shear, and the base-structure amplification fundamental period.

Other Seismic Considerations:

a) Liquefaction. Liquefaction is the condition where the seismically induced pore pressure exceeds the strength of the soil, causing the soil to behave as a liquid. Material that is susceptible to liquefaction is unsuitable as foundation material, backfill, or embankment dam material. However, if the designer is faced with this condition, the foundation treatment should be referred to a qualified geotechnical engineer for evaluation. In general, material subject to liquefaction are loose, uniformly graded sands and silts.

b) Soil Structure Interaction. Structures of soil foundations have to consider the effects of soil structure interaction.

c) Dynamic Silt: another seismic soil consideration is dynamic silt or dynamic soil loading.

   The dynamic silt may be treated as an equivalent fluid and loading, determined in the same manner as hydrodynamic loads.

d) Vertical Acceleration. Earthquakes generate forces traveling in all directions. The designer must develop an appropriate vertical inertial force based on the site, the structure, and local requirements.

Water Retention Structures:
Water conveyance system which is the part of the hydroelectric project components are generally designed with the same seismic criteria as the dam itself.

Seismic loadings should be selected after considerations of accelerations which may be expected at each project site as determined by the geology of the site, proximity to major faults, and seismic history of the region as indicated by seismic records. The seismic parameters for the design of Modi Khola HEP is considered to be 0.1 x g in the vertical direction.

Ice Loads

The exposed conveyance system in high mountains where ice is formed in canal, desanding basin and forebay, designer should consider ice loads to the respective structure. Ice pressure is created by thermal expansion of the ice and by wind or current drag. Pressures caused by thermal expansion are dependent on the temperature rise of the ice, the thickness of the ice sheet, the coefficient of expansion, elastic modulus, and strength of the ice. Wind drag is dependent on the size and shape of the exposed area, roughness of the surface, and the direction and velocity of the wind. Ice loads are usually transitory. A second aspect of ice loading is its contribution to vertical live loads. If exposed decks or platforms ice-up, in addition to snow, the weight of an assumed amount of ice should be included in the design.

Wind and Snow Loads

Areas which experience extreme loading should use the locally accepted code loadings.

Equipment Loads

Items to consider for equipment loads are:

- Gate loads – partially open gates, fully closed gates, vibration, down-pull, opening and closing forces, and hoists.
- Trashrack loads
- Clogged trashracks and resulting differential head
- Crane loads – impact, crane weight plus load

3.2.1.5 Construction Materials

General

A variety of materials can be used in hydropower development. The construction and economics generally dictate which materials are to be used. Probably the most widely used material is reinforced concrete because of its strength and durability characteristics, and its environmental neutrality. Materials which may be considered for structural design are : cast-in-situ concrete, pre-cast concrete, pre-stressed concrete, post-tensioned concrete, concrete crib walls, masonry, steel, timber, plastic, protective coatings, construction joint materials, etc. All materials used in the construction of hydroelectric project should be manufactured and tested in strict accordance with the appropriate acceptable specifications and related design specifications.

Materials should be new and of first quality, and should conform to the appropriate design code or specification

Types and Uses of Materials

Reinforced Concrete:
Reinforced concrete is the most commonly used construction material at the conveyance system for canal, desanding basin, spillway, forebay, concrete lining of tunnel, etc. Generally, conveyance systems are found in aqueous environments which pose moderate corrosion potential. Providing adequate concrete between the reinforcing steel and the water normally minimizes the potential corrosion. Also, exposed conveyance systems are often noted for their mass to provide adequate stability. Concrete is a dense, tough, structurally adaptable, and economic material.

Un-reinforced Concrete:
Un-reinforced concrete should not be used in a hydropower facility, except in mass concrete applications. In this use, the designer should consider the need for thermal controls on the placed concrete and provide sufficient expansion/contraction joints so as the structure cools and cracking is controlled.

Differential settlement and the potential for seismic activity, although remote in some areas, making of un-reinforced concrete walls, beams, columns, slabs, etc is undesirable from a structural and water tightness perspective.

Shotcrete:
Shotcrete consists of cement, sand, pea gravel, water, and admixtures. In many respects, shotcrete is like concrete without the coarse aggregate. Shotcrete is pneumatically applied to a surface, usually rock or soil, to improve ground stability. For open excavation and tunnel portal work, shotcrete and mesh can support the in-situ material by locking blocks of adjacent rock masses together to reduce the potential or rock falls. It also protects rock materials from continued weathering and corrosion. Shotcrete is applied by one of two processes- wet-mix or dry-mix. When the batch is dry-mixed, water is added at the nozzle along with any other admixtures. When the entire batch is mixed 'wet" and shot from the nozzle, the process is referred to as wet-mixed. The primary difference is when the water is applied to the other ingredients.

Masonry:
Masonry comes in many styles and configurations. Most common are concrete block and slay brick and river bed materials. Masonry, like concrete, has numerous structural applications like construction of canal, desanding basin and forebay and other components of hydropower structures. Its strength is primarily compressive in nature; however, by including fully grouted reinforcing steel, bond-beams, and structural diaphragms, lateral load resisting systems can be developed.

Structural Steel and Miscellaneous Metals:

a) Carbon Steel: Carbon steel is commonly used as a secondary structural material in structural design. Steel plate and shapes are manufactured to a variety of material and strength properties.

b) Stainless Steel: Stainless steel is similar to carbon steel with more chromium. A typical application is where coatings are uneconomic. Surfaces which are subject to mechanical wear, such as rolling or sliding surfaces, may be made of stainless steel. Gate sealing surfaces are almost always stainless steel and sometimes bronze. Where flows create excessive turbulence and/or cavitations, the designer should also consider stainless steel. Aggressive corrosion (water, soil) should be evaluated by a metallurgical engineer and the appropriate materials selected.

c) Rock Bolts, Rock Anchors and Soil Nails. Rock bolts, rock anchors and soil nails are used to support ground excavations and sometimes to anchor the structure to the foundation surface.

Timber
Timber construction is typically limited to structures where the exposure to environmental deterioration is slight. Roof framing of concrete or masonry buildings is the most common use of wood. Some species of woods have some natural resistance to "dry-rot" or animal consumers like termites and beetles. Otherwise, timber requires a more positive protection from deterioration such as pressure treating with a preservative, metal flashing, and/or paint.

Earth and Rockfills
Earth and rockfills are commonly used as backfill material. The backfill should be graded to meet the structural requirements. For example, if the bearing wall of the structure is designed to resist drained lateral earth pressures, then the backfill should be free draining or a drain system included. Typically the amount of material passing a #200 sieve should not exceed 11 percent to be considered free draining.

When the backfill is being placed and compacted, ensure that the backfill is brought up on small lifts consistently on both sides of the structure so as not to unbalance the lateral loads. If the designer considers 100 percent of the structures' mass to provide stability, there may be short-term constructions when the backfill is complete. But the incomplete structure cannot resist the full sliding forces. The designer is encouraged to monitor the construction during any critical phases to advice the contractor on appropriate sequences.

Plastic
Plastic trash rack bars may offer improved performance in cold weather climates. The thermal conductive characteristics of plastics reduce the potential for the formation of frazil ice on the rack but in general practice in Nepal, these materials are not used for the trash rack purposes.

Protective Coatings

Protective coatings are typically associated with steel construction; however, concrete sealants are becoming more prevalent. Coatings may consist of zinc rich paints, epoxies, urethanes, galvanic compounds, and coat tars. Environmental regulations are affecting the production and applications of many protective products. Products containing volatile organic compounds (VOCs) and carcinogens should be reviewed carefully with local, state and federal agencies before being specified on a project.

Serviceability Considerations

The performance requirements for the structure should be considered in the selection and configuration of the structural system and materials. While developing the design criteria, the engineer should evaluate the project's requirement for sustained operation following a design event such as a flood, snow storm, or earthquake. The structure should be designed to survive the least likely environmental event with an anticipated design life in excess of 50 years. Floods, design based earthquakes, windstorms, hail, and snow storms all contribute to the design criteria. Where the failure of a structure does not affect life safety, the engineer may consider designing to a lower return interval event after exhausting other relevant design guidelines.

Material Specifications

Concrete consists of the proportioned mixing of cement, aggregates, water, and admixtures to produce a concrete which is in conformance with the requirements of the design and the serviceability requirements of the structure. Normal Strength Gain Concrete achieves its minimum specified strength within 28 days and its other properties are in conformance with the requirements of the reference standards. High Early Strength Gain Concrete achieves its minimum specified strength gain at an accelerated rate, usually between three to ten days. Concrete can be designed by several methods, ACI's Reinforced Concrete Design for Buildings, Publication 318, presents the ultimate strength design method. The U.S. Army Corps of Engineers uses Strength Design for Reinforced-Concrete Hydraulic Structures, (EM 1110-2-2104 USCOE, 1992) as its concrete design specification.

Cement:

Various types of Portland cement for concrete construction could be selected. Each cement type has specific properties benefiting with the specific design conditions. For example, if the aggregates are slightly reactive, cement should counteract the reactive property. For mass concrete placements, the amount of cement should be minimized or "low heat" cement should be used.

Silica Fume:

Silica fume can be used to produce concrete with very low porosity, enhancing water tightness, and potentially increasing strength. Silica fume is a waste product from the Ferro-silicon manufacturing process, and reacts with the lime liberated during the hydration of Portland cement. Low porosity is achieved due to the smaller particle sizes of the silica fume (compared to Portland cement), and to the low water/cement ratio of the mix. A super plasticizer is typically used to obtain a suitable workability.

Pozzolan:

Pozzolan is often used to reduce the heat of hydration of mass concrete applications. Pozzolan materials are typically volcanic ash, pumicite, opaline shales and cherts, calcined diatomaceous earth, burnt clay, and fly ash. The most common artificial pozzolan is fly ash, or pulverized fuel ash, which is obtained by electrostatic or mechanical means from the flue gases of furnaces in coal-fired power stations. The pozzolan, is finely divided form and in the presence of moisture, will chemically react with the lime (liberated by hydrating Portland cement) at ordinary temperatures to form compounds possessing cementious properties.

Aggregates:

Both coarse and fine aggregates should be clean, sound, non-reactive and free of deleterious coatings, silts, clay wood and organic materials. Non-reactive aggregate quarry or source should not contain any materials that are deleteriously reactive with alkalis in cement in an amount sufficient to cause expansion in mortar or concrete.

Water:

Water used to batch concrete should be clean and free from deleterious amounts of silt, oil, acids, alkali, salts, organic substances and contain no chlorides.

Admixtures:
Admixtures are products added to the concrete to enhance the workability of plastic concrete, reduce water requirements, improve strength gain, improve durability and wear resistance (hardness), aid in curing, impart waterproofing properties while concrete is in its plastic state, enhance dispersion of the cement particles, and improve the entrainment of air. Admixtures include both products that are added and mixed prior to concrete placement as well as products that are added after the concrete has been placed and is still in a plastic state. Concrete bonding agents, sealants and waterproofing agents that are applied after the concrete has hardened are not defined as an admixture. All products should conform to applicable standards and with industry standards. All combinations of admixtures that will be added into any batch should be compatible, and when mixed in accordance with the manufacturer's instructions, should not produce any adverse side effects to the concrete.

Air Entraining Agents:
Air entraining agents may be used to improve concrete's durability when exposed to extreme weather variations.

Water Reducing Agents:
A water reducing agent may be used to increase the workability of the mix without decreasing the strength.

Retarder:
A water reducing/retarding agent may be used to retard to concrete set. The agent should be used in the amounts recommended by the manufacturer to obtain the desired retardation. Normal slump and air contents specified should be maintained during the use of the set retarder admixture.

Accelerators:
Accelerators may be used to decrease concrete cure times and/or provide early high strengths. Accelerators should not contain calcium chloride and should not increase the drying shrinkage or be detrimental to the reinforcing material.

Super Plasticizers:
High range water reducing admixtures may be used to develop mix designs with improved workability. However, super plasticizers should only be used on a case-by-case basis.

Pigmentation:
Where required by environmental concerns, concrete structures can be "painted" to blend with the surroundings. A pigment can be added to the concrete to produce the desired coloring. Colored concrete typically fades with exposure to ultra violet rays (sunlight) and may even appear blotchy or uneven. Determining the appropriate color may prove to be an involved and lengthy negotiation with the involved parties. Unless very specific quality controls are used, the seasoned color of the concrete may not be the same color originally anticipated. If a uniform color is required, then tight controls on the pigment and the cement must be anticipated. Typically only white cement has a uniform color because of ASTM requirements; hence adding controlled pigment to white cement ensures a uniform concrete pigment.

Finishes and Sealants

Joint Filler and Joint Sealer:
Expansion and isolation joint fillers should be pre-molded strips of Styrofoam or other specific joint filler. There are a variety of commercial products available to create seal and maintain expansion joints.

Waterproofing:
Generally, conveyance system structures are submerged and the occasional dewatered condition does not require completely dry interior conditions. However, where water-tightness may be a concern, all exterior surfaces of the structure should be applied with a waterproofing compound. Placing the waterproofing compound on the exterior surface of the concrete will minimize the amount of water which permeates into the concrete and reinforcing steel. Interior placement of the compound will improve moisture resistance and reduce "sweating" of the walls.

Roughening Agent:
For concrete steps, stairs, and other traffic areas subject to slippery conditions, an abrasive agent abrasive, such as aluminum oxide, can be applied to the concrete surface during finishing improving traction.

Curing Agent:
A curing compound which dries clear should be used for all exposed surfaces. No wax-based product or other type of product that would prevent the proper bonding of patches, paint, mortar, or plaster should be used.

Reinforcing Steel or Rebar

Reinforcing steel or rebar (sizes, shapes, corrosion resistance), comes in many different sizes (diameters) and strengths. Reinforcing steel can be deformed billet-steel bars (ASTM A615-S1), welded wire fabric cold drawn wire (ASTM A 82); welded sire steel (ASTM A185); and welded deformed steel (ASTM A497), The selection of a particular type of reinforcing steel should consider the complexities of the reinforcing steel patterns, accessibility, and ease of installation, in addition to strength requirements.

Structural steel

Steel design and fabrication should conform to the requirements of the Code referred for the structural design.

Timber

Timber design and fabrication should conform to the requirements of the Code referred for the structural design.

Masonry

Like timber, masonry is used throughout the building industry and has many available design codes which are applicable. Masonry construction uses many of the specifications of cement, water, reinforcing steel, and fine aggregates typically found in concrete construction.

Rock Bolts, Rock Anchors, and Soil Nails

Rock bolts, rock anchors, and soil nails are used in surface excavations of rock and soil reinforcement and may be tensioned or intentioned (dowels). All bolts should be steel, whether smooth or deformed, and grouted in a tensioned or intentioned state. Rock bolts, rock anchors, dowels, cement grouts, and resins should be produced by a manufacturer regularly engaged in the production of the specified items with a history of successful installations.

Formwork

The following items should be considered when laying out the structure and presenting the design requirements on the engineering drawings.

Chamfer Strips:
Chamfer strips should be placed in forms to bevel all edges and angles of concrete. The lack of sharp 90-degree corners will minimize the amount of broken edges and irregular "vertical" lines of the structure. Chamfer sizes vary by orientation – exterior, vertical wall corner chamfers are typically larger than those for penetrations or openings which have no embedded steel framing and for small equipment pedestals. Chamfer sizes range from ¾ inch by ¾ inch to 1-1/2 inch (approximately).

Metal Forms:
Reusable metal or steel forms are sometimes used where lifts of the same height and configuration and placed repeatedly. The designer should consider repeatedly using cross sections and wall sizes where forms might be economically re-used.

Architectural Form Liners:
Where the finished concrete surface will be textured, the designer may consider using form liners. Rigid plastic liners are attached to the formwork prior to placing the concrete. Many of the same considerations as for metal forms should be used during design.

Leave-In-Place Forms at Construction Joints:
In complicated structures, there are sometimes benefits to staging concrete placement for blockouts, first and second stage concrete, pedestals, and where a vertical formed construction joint is used. In lieu of using a smooth form, brush finish or "green cutting" a leave-in-place form can be considered. The material resembles light gauge expanded galvanized metal which s easily molded and cut with sheet metal shears. When concrete is placed against the leave-in-place form, cement paste and sand extrudes through the form to create a rough texture with approximately 1/4 –inch to 3/8- inch profile. The leave-in-place form can remain in place or be removed.

Material Testing Requirements

All material and fabrication should be of the highest quality. Any damaged material should be rejected, removed, and replaced; the material and fabrication should meet the governing acceptance standards.

3.2.1.6 System Analysis

Stability

Water Retention Structures:
The following load combinations are considered for the evaluation of hydropower structures

a) Usual/Normal operation.
b) Unusual/Flood operation.
c) Unusual/Ice. Consists of normal operating loading plus ice, if applicable.
d) Extreme/Normal plus EQ. Normal operational loading plus the inertial force due to the seismic acceleration of the structure, and the increased hydrostatic forces due to the water retention.
e) Extreme/Construction Plus EQ Construction of the conveyance system like desanding basin/canal/forebay is completed, with no water inside, but the soil on the outer wall are saturated and the earthquake acceleration force is applied on the structure.

Analysis

Moment Equilibrium Analysis:
Allowable stresses for concrete and foundation martial may be determined by dividing the ultimate strength by the factors of safety.

A hinge is needed for an overturning analysis to be valid. When the base is cracked, foundation cohesion disappears and uplift increases. It is extremely unusual for the center of loading to be far enough above the center of gravity of a structure to create an overturning a moment without sliding occurring first. This needs to be checked on a case-by-case basis.

Shear Friction analysis:
The gravity method of analysis includes a two-dimensional calculation of factors of safety for sliding, location of the resultant of all forces, and determination of maximum stresses along postulated failure planes.

The analysis assumes that the structure is a homogeneous, isotropic, and uniformly elastic material. All loads are transmitted to foundation or planes within the structure through cantilever action of the structure without support from adjacent monoliths, and normal stresses are distributed linearly on horizontal planes.

The factor of safety for sliding stability on a horizontal plane is:

$$\frac{CL + (W-U)\tan(\phi)}{\Sigma H} \quad \text{\_ \_ \_ \_ \_ \_ \_ \_ \_ \_ \_ \_ \_ \_ \_ \_ \_} \qquad (3.96)$$

Where

- C = cohesion
- L = base length in compression
- W = sum of vertical-down forces
- U = sum of vertical-up forces
- $\tan(\phi)$ = coefficient of internal friction
- ΣH = sum of horizontal driving forces

Floatation analysis:

The floatation factor of safety is the ratio of the vertical-down forces divided by the vertical-up forces on the structure. The down-force may be comprised of the structures' dead weight, backfill material, and water above and contained within the structure. The up force may consist of uplift pressures on the structure and the vertical seismic component. Structures can be susceptible to floatation problems if little mass concrete is used for the structure.

Factor of Safety for Floatation = W/U

Where:

- W = sum of the vertical-down forces
- U = sum of the vertical-up forces

Finite Element Analysis:

Numerical modeling techniques have been improving at a rapid pace. As the personal computer gets more powerful, the use of numerical modeling for hydraulic structure analysis will become more common. Finite element models can produce extremely accurate results, and are the preferred method to analyze dynamic response of structures in earthquakes in Seismic Zones; however, there are some drawbacks with this method. Limitations of finite element modeling include the expense of programs, difficulty in checking the results, and the requirement of the knowledge of the limitations of the modeling techniques.

Factors of Safety

In most cases, the FERC guidelines will govern; however, if the structure is under the jurisdiction of the US Army Crops of Engineers or the U.S. Bureau of Reclamation, there requirements may govern. The FERC classifies its dams into one of three categories based upon exposure to loss of life and economic loss: Low, Significant and High. The definitions may be found in the FERC's Engineering Guidelines (1991).

FERC Recommended Safety Factors for Moment Equilibrium

Loading Conditions	Factor of Safety Overturning		Factor of Safety Sliding	
	High Hazard	Low Hazard	High Hazard	Low Hazard
Usual	3.0	2.0	3.0	2.0
Unusual	2.0	1.5	2.0	1.25
Extreme	>1.0	>1.0	>1.0	>1.0

These factors of safety are applicable for determining the allowable unit stress of concrete and foundation material by dividing the ultimate stress for the material by the appropriate factor of safety (listed above) and determination of the sliding factor of safety for cases without extension state-of-art foundation exploration.

U.S. Army Corps of Engineer Criteria:

The U.S. Army Corps of engineer used several criteria of evaluation of hydroelectric projects. Existing concrete structures are covered under Engineering Manual 110-2-256 (USCOE, 1981). This divides the structure being investigated and the surrounding soil mass into a series of blocks, and analyzes it in a manner similar to the slope stability program. The limit equilibrium analysis has been incorporated into a computer analysis package (CSLIDE) Available from the U.S. Army Corps of Engineers. The

FERC Engineering Guidelines (FERC, 1991) reference ETL 1110-2-256 in its description of Internal Hydrostatic Load (uplift), and in its evaluation of sliding stability. The advantage with computer analysis is that the program searches for multiple planes of weakness within the structure and is, therefore, not limited to an assumed geometry of failure.

The factors of safety for major concrete structures in ETL 1110-2-256 are zero for normal static loading conditions (usual case and 1.3 for seismic loading conditions extreme case). The FERC Engineering Guidelines (FERC, 1991) stipulate that the factors of safety for sliding in the ETL 1110-2-256 (USCOE, 1981) are based on the premise that extensive foundation exploration and testing has been conducted using sophisticate, state-of-the-art techniques. In order to use the factors of safety in ETL 1110-2-256 (USCOE, 1981), FERC geotechnical engineer should be consulted concerning the adequacy of he foundation exploration and testing program. In general, the factors of safety will govern. Flood walls and retaining walls are covered in EM 1110-2-2501 (USCOE, 1948), and EM 1110-2-2502 (USCOE, 196).

U.S. Bureau of Reclamation Criteria

Loading Condition	Factors of Safety			
	Major Structures		Minor Structures	
	During Construction	Structure Completed and Equipment Operating	During Construction	Structure Completed and Equipment Operating
Normal	2.5	3.5	1.5	2.0
Extreme	1.1	2.0	1.1	1.5

Base Stress

The stresses in the base material will be determined as a result of the stability analysis. The allowable stress for concrete foundations is based on the mix design and concrete testing results.

Foundation

Rock Foundations:

a) Foundation Stresses: The allowable stress capacity of the foundation material is optimally determined by laboratory testing. References are available that give general indications as to the allowable stresses in various types of foundation materials.

b) Zones of Weakness and Stability. It requires that rock foundations be analyzed for stability if there is a potential for direct shear failure, or whenever sliding is possible along joints, faults, or shears. Sliding failure may result when the rock foundations contain either discontinuities or horizontal seams near the surface. Anchoring the rock mass or additional rock excavation may be required.

c) Zones of Weakness and Foundation stresses: In addition to stability, potential for overstressing the structure foundation exists if there are zones of weakness which allow for differential displacement of rock blocks on either side of the weak zones, or when the weak zone represents an excessive span for the bridge. Such zones should be strengthened during construction.

Soil Foundation:

a) **Seepage**. In addition to bearing capacity of a soil foundation, structures founded on soil need to be checked for the seepage potential. The structure requires that the foundation hydraulic gradient be checked for seepage potential using the weighted creep method. If the seepage potential exceeds the allowable level, then a flow net analysis can be used to determine the exit gradient at the toe. A safety factor greater than 2.0 against piping at critical points in the foundation is acceptable.

b) **Pile Foundations**. These structures are examined on a case-by-case basis. For conveyance structures, it is common to have a large component of load in the lateral direction. Items to be considered include the combined stresses in the pile from lateral and vertical loads, the stresses in the soil, and deflections. Conservative methods are available to make reasonable estimates of pile lateral load capacities and deflections. Given the complexity of the situation, it is often advisable to use battered piles to account for horizontal loads. Computer programs are available for analyzing more complex pile group configurations. More accurate analyses are available for pile group systems that make use of computers.

Seepage Control Methods:
Various methods are available to control seepage beneath the structure. The, application depends on the type of foundation material and the anticipated effectiveness of the seepage control. For example, isolated injection grouting may be effective in fractured rock. The following seepage control methods are generally practiced:

- Cutoff apron
- Sheet pile, concrete, or other upstream or downstream cutoffs
- Downstream apron, with scour cutoff
- Grout curtains
- Drainage system for uplift relief

3.2.1.7 Component Analyses

Load Combinations

In general, the loading combinations used for the design of structural components will depend on the structural element and expected usage. Engineering judgment is needed to determine the appropriate loading cases for the particular element. In general the following load cases are investigated:

- Dead plus floor live plus roof live (or snow)
- Dead plus floor live plus wind (or seismic)
- Dead plus floor live plus wind plus snow/2
- Dead plus floor live snow plus wind/2
- Dead plus floor live plus snow plus seismic
- Dead plus hydrostatics pressure

Method and Reference for Analysis

The method of analysis and design procedures for structural components are usually provided by codes of standard practice, such as the America Concrete Institute, American Institute of Steel Construction, the Post Tensioning Institute, IS code, British code etc. The standards of practiced may be modified for the special application to hydraulic structures. Some codes practiced in Nepal are:

- IS : 456-1978 : Code of Practice for Plain and Reinforced concrete.
- IS : 1893-1970: Criteria for earthquake resistant design of structures
- BS 8110:1985: Structural Use of Concrete
- BS 8007: 1987: Design of reinforced concrete structure for retaining aqueous liquids
- BS 6031: 1981: Code of practice for earthquake
- IS 226: 1984: Code of practice for general construction in steel

Special Components

For many of the special components such as trashracks, gates, and valves, the structural design is performed by equipment vendors or fabrication shops. In this case, the designer has the responsibility for preparing performance specifications and checking shop drawings prepared by the manufacturer to ensure that the equipment satisfies the contract requirements.

Trashrack and Frame:
Items to be considered in trashrack design include vibration of the bars, partial clogging of the bars with debris, ice loads, and loads induced by trashrack cleaning machines.

Design criteria for the trashrack and supporting frame differ from organization to organization and project to project. One approach assumes that a percentage of the trashrack area becomes clogged with debris. Accordingly, the trashrack is designed for a differential water pressure equivalent to the percentage of the water height. The percentage varies from 50 to 100 percent. Because the supporting frame is typically embedded in the structure and difficult to replace, the frame is designed for twice the differential pressure of the trashrack. A second approach assumes a 1.5m differential water pressure on the trahsrack and a 3.0-m differential on the frame. The designer is encouraged to investigate the potential debris loading at the structure site and includes the implications in the design.

Trashrack frames should be designed to allow for future maintenance or replacement of the trashracks without excessive difficulty or danger to personnel. Examples of this would be to design the trashrack in removable sections that could be detached and replaced with a minimum of special requirements, such as underwater cutting or welding, or using large barge-mounted cranes.

Stoplogs and Gates:
Design considerations for stoplogs and gates can be found in various industrial publications.

3.2.2 Structural Analysis Specific to Penstock

3.2.2.1 General

The design, manufacturing, installation, testing & commissioning of penstock pipe should be carried out in conjunction with the design of the civil works, hydraulic steel structures and power house equipments. The basis for design will be a set of design criteria, which shall ensure that the penstock shall meet stringent safety standards and confirm to the associated civil, electromechanical and transmission structures.

3.2.2.2 Loadings

The loadings that must be taken into account in designing penstocks should include, but not limited to the following:

- Internal and external pressure
- Impact loads; including rapidly fluctuating pressure such as surge chamber water surface fluctuations (surge) and water hammer
- Dead weight of steel, concrete, and water
- Miscellaneous loads such as other components, operating equipment, insulation, corrosion resistant or erosion-resistant lining, valving, and piping
- Seismic load, wind loads, snow loads, and vibration loads where applicable
- Reactions of supporting legs, ring girders, saddles, or other types of supports
- Temperature effects
- Reactions caused by water impingement: changes in direction, changes in diameters and loads from valves
- Construction loads
- Backfill loads

The penstock or portions thereof may be exposed to more than one pressure, temperature and mechanical load condition as provided in the design criteria. The specified design parameters for design loadings shall be called design pressure, design temperature, and mechanical design loads.

Pressure: The internal design pressure shall not be less than 100 percent of the maximum internal pressure under normal operating conditions. This includes surge and water hammer. The external design pressure shall not be less than 100 percent of the maximum external pressure.

Temperature: The following temperature considerations should be evaluated for design.

- Any external or internal heat generation effects shall be incorporated in the establishment of the design temperature.

The design temperature shall not be less than the difference between the steel temperature at the maximum daytime's temperature and the minimum night time temperature. The steel temperature under direct sunlight can be much greater than the maximum daytime temperature. The extreme temperature range can occur during construction or during plant outages when the penstock is drained.

Design and Service Loads: Design and service loads vary, but the most important load imposed on a penstock system results from hydrostatic and hydrodynamic internal pressure. Other key load conditions include external pressure loads, gravity-related loads, loads resulting from temperature changes, and seismic, wind, and snow loads; also included are loads from vibration and cyclical pressure fluctuation caused by plant operations and turbine equipment. The designer must identify and quantify these loads. Service levels and load combinations recommended in this may need to be augmented to include other load combinations more critical to a specific installation. Cyclical water-hammer pressure fluctuations must be considered in determining the potential for in-service cracking. The effects of temperature-related loads must not be underestimated, particularly for exposed penstock sections and restrained shell configurations, and in the development of secondary stress conditions and brittle fracture. Designs based on selected service levels can be checked for additional loads by the method of superposition or other acceptable methods.

Factors of safety and resulting allowable stresses must be specified, taking into consideration both the yield and tensile strength attributes of the material. Other critical steel properties to be considered include ductility, resistance to brittle failure, and weldability. A wide range of pressure vessel steels are available for use in penstock design. These steels exhibit varying ranges between the yield strength and the tensile strength for the steel. For economy of design, localized stress conditions exceeding allowable should be considered for acceptance, given the probability of local yielding and the redistribution of stresses.

The designer must be aware of possible special loads that may be imposed on the structure, particularly during construction, filling, dewatering, and testing. Stresses developing from these special loads must be determined and adjustments made to accommodate unacceptable stress levels. As stated earlier, special consideration must be given to stress conditions that can develop within the structure during hydrostatic testing and to the configurations for supporting the structure during testing. It is recommended that the designer develop the test procedure as well as the preferred method for supporting the structure during testing.

3.2.2.3 Selected Concept

The selected penstock configuration should incorporate material and designs that minimize life cycle costs, with prudent consideration given to technical, environmental, constructability, and maintainability issues.

3.2.2.4 Definitive Design

The definitive design phase consists of compiling final design-related data, finalizing the conduit alignment and layout, and confirming the final plant operating characteristics. Turbine, generator, governor, and closure valve characteristics that influence water hammer analysis also must be determined.

Definitive guidelines and design criteria must be prepared for:

- Physical layout, conduit alignment, supports, and anchor blocks
- Material selection for the penstock, tunnel liners, and appurtenances
- Design and service loads
- Special considerations for manufacturing and field installation

3.2.2.5 Materials

The following factors decide which material to use for a particular project:

- Size of the plant
- Design pressure
- Method of jointing
- Diameter and friction loss
- Weight and ease of installation
- Accessibility of the site
- Terrain
- Soil type
- Design life and maintenance
- Weather conditions
- Availability
- Relative cost
- Transport to site

The following materials are used for the penstocks of different size plants:

- Steel
- Unplasticized polyvinyl chloride (UPVC)
- High density Polyethelene (HDPE)
- Medium density Polythelene (MDPE)
- Ductile iron
- Asbestos cement
- Prestressed concrete
- Wood stave
- Glass Reinforced Plastic (GRP)

The purpose of this section is to assist in the selection of the proper materials for the design and construction of penstocks and tunnel liners. This section only covers the use of steel in the manufacture of penstocks. The use of plastic, fibreglass, wood staves and concrete in regards to material properties is not discussed. The specific manufacturers of these materials should be consulted for their properties, performances and uses for pressure system.

Quality

All steel used as base material in the fabrication of penstocks are to be manufactured and tested in strict accordance with appropriate ASTM or other specifications. Penstock and tunnel liner shells are to be fabricated using steel plates of pressure vessel quality unless the design loadings do not control the plate thickness. In these cases, lower quality steels can be considered. Ring girders, stiffener rings and support systems are to be fabricated from plate of structural shapes produced from structural quality steels.

Steel to be used for the fabrication of penstocks of a hydro-electric project should meet the following requirements:

a. It should stand against maximum internal pressure including dynamic pressure,

b. It should stand against frequent dynamic change,

c. It should have required impact strength to be able to deform plastically in the presence of stress concentrations at notches and bends.

d. It should have good weld-ability without preheating, and

e. It should not require any stress relieving after welding.

The requirements (a) to (c) are essential while requirements (d) and (e) are preferable.

Allowable Stresses

The allowable stresses and the factor of safety to be adopted depend upon the yield point stress and ultimate tensile strength of the steel, loading condition and the location where steel lining is provided. Following allowable stresses should be adopted in design of the steel penstock:

(a) In normal operating condition, the design stresses should not exceed one-third of the minimum ultimate tensile strength or 60 percent of minimum yield point stress of steel, whichever is less.

(b) In intermittent condition, the design stresses, should not exceed 40 percent of the minimum ultimate tensile strength or two-thirds of minimum yield points stress of steel, whichever is less.

(c) In the emergency condition, the design stress should not exceed two-thirds of minimum ultimate tensile strength or 90 percent of minimum yield point stress of steel, whichever is less, and,

(d) In exceptional condition, the design stress should not exceed the minimum yield point stress.

When rock participation is considered in the design, the stresses in steel lining under normal loading condition without rock participation should also be checked and should not exceed two-thirds of minimum yield point stress or two-thirds of minimum ultimate tensile strength, whichever is less. In intermittent and emergency conditions of loading it should not exceed the minimum yield point stress.

Loading condition should be considered as given below:

(a) Normal condition includes static head along with pressure rise due to normal operation or head at transient minimum surge, whichever is higher.

(b) Intermittent condition includes those during filling and draining of penstocks and maximum surge in combination with pressure rise during normal operation.

(c) Emergency condition includes partial gate closure in critical time of penstock at a maximum rate, and the cushioning stroke being inoperative in one unit.

Exceptional condition includes slam shut, malfunctioning of control equipment in the most adverse manner resulting in odd situation of extreme landing. This should not be taken as design criteria.

3.2.2.6 Certified Design Criteria

Penstock must be designed in accordance with the requirements of the Certified Design Criteria (CDC), which must be prepared by a qualified, experienced registered professional engineer who is knowledgeable in penstock design. The certified design criterion gives the penstock designer the following design input information:

- internal and external pressure profiles
- dead loads
- externally applied mechanical loads
- live loads
- wind, snow, and seismic loads
- thermal loading conditions, and
- cyclic loading conditions including dynamic loads

The above described loads are combined to describe the various load combinations for the following conditions:

- normal operating conditions
- intermittent conditions
- emergency conditions
- exceptional conditions
- construction conditions
- hydrostatic test conditions.

3.2.2.7 Shell Thickness of Penstock Pipe

For internal pressure, the pipe shell thickness is given by the relation:

$$t = \frac{PR}{f\eta} + \varepsilon \qquad (3.97)$$

Where,

t = Thickness of pipe shell in cm
P = Internal pressure in kg/cm^2
R = Internal radius of pipe in cm
f = Allowable stress in kg/cm^2
η = Welding joint efficiency 0.9
ε = Corrosion allowance 0.2 cm

Minimum plate thickness of pipe shell is to be calculated considering handling and transportation for the case where calculated thickness using relation (Used in Kulekhani-I Design report) for internal pressure is very small.

$$t = \frac{D + 800}{400} \qquad (3.98)$$

Where,

t = Minimum thickness of pipe shell in mm
D = Internal diameter of pipe in mm

3.2.2.8 Penstock Vibration

Water merely flowing through the penstock does not cause the penstock to vibrate. However, if a pressure wave is generated, the penstock may vibrate. If, by chance, the frequency of vibration of the pressure wave is very close to the natural frequency of vibration of the penstock itself, the state of resonance may be set up and excessive vibration of the penstock can be observed. From the purely analytical point, if both frequencies coincide exactly, the penstock will vibrate excessively.

If the penstock is to be designed without knowing these parameters, the designer should be somewhat conservative in the design.

When a penstock vibrates excessively during the operation of the hydroelectric plant and the vibration reaches such a degree as to be disturbing the smooth operation of the station (power swings), by visual means or by stress level (strain gauges), counter measures should be taken promptly to eliminate the causes of vibration.

The Marsyangdi Hydropower station operating in Nepal had problem of penstock vibration due to the problem in branching in one unit. This problem has been rectified afterward by proper reinforcement of the trifurcation junction.

To determine appropriate measures to deal with the vibration, it is necessary to find the source of the pressure variations responsible for the vibration and it should be eliminated. The causes of the vibration are:

- Rotation of water turbine
- Number of vanes of the runner
- Draft tube whirl

When elimination of the cause of vibration is not possible, the natural frequency of the penstock should be changed by adding reinforcing material on the penstock, providing additional supports or anchor blocks or encasing the penstock in concrete.

3.2.2.9 Type of Installation

Three types of installation have been discussed earlier; Type-wise design guidelines for each type of installation will be provided hereunder.

Exposed Penstock (Aboveground)

Penstock Shell Design: The main factors that govern the required shell thickness are:

- Thickness required for shipment and handling;
- Thickness required to resist the imposed loads, considering the appropriate allowable stresses

Additional factors in determining shell thickness include:

- Acceptance criteria for mill and fabrication tolerances
- Criteria for corrosion allowance, if elected in lieu of coating and lining
- In addition to minimum shell thickness for handling, the shell thickness, support types, and support spacing must be selected so that the maximum deflection of the pipe filled with water, acting as a beam between the supports does not exceed 1/360 of the span
- Similarly, to avoid pipe buckling due to full internal vacuum, the D/t ratio should be not less than 158. If the ratio is less than 158, stiffeners may be required.

Minimum Thickness for shipping and Handling: The minimum thickness of the penstock shell for shipping and handling can be calculated by using following relations:

$$t_{min} = D/288 \qquad (3.99)$$

(Pacific Electric's & Gas formula)

$$t_{min} = (D+20)/400 \qquad (3.100)$$

(Bureau of Reclamation Formula)

Where,
t_{min} = minimum thickness in mm and

D = pipe diameter in mm

Calculated larger value should be adopted.

Internal Pressure Design for Free-Standing Penstocks: The minimum required plate thickness should be computed considering the maximum pressure rise due to full load rejection on all penstock units operating at full or partial gate discharge. Turbine wicket gates are assumed to close in the normal governor closure time during load rejection. The envelope of the maximum pressures that occur over time along the penstock is generally assumed to vary linearly from the maximum pressure head due to water hammer at the units to the maximum surge at the surge tank in the absence of a surge tank. It varies linearly to the reservoir surface at the intake. The pressure rise due to full closure of a turbine guard valve or a bypass pressure relief valve should also be investigated. Guard valve closure times are normally set to produce less pressure rise than a turbine wicket gate closure.

Load acceptance always must be investigated to determine that the hydraulic grade line does not drop below the penstock profile at any point. This can result in a separation of the water column. Internal pressure drops to the vapor pressure of water in separation area, and a potential for collapse due to external air pressure exists. In addition, the rejoining of the water column could cause excessive and undesirable water-hammer pressures.

Hoop stress for penstock Design: The pipe shell thickness calculation for internal pressure is mentioned earlier in Section 3.2.2.7. Conservative values must he used for allowable hoop stress, depending on the type of steel shell material strength characteristics and the most likely factors of safety that will be considered for the installation type.

The internal design pressure must be at least equal to the maximum static head at the point of the penstock being investigated. Some refinement to this is possible by performing a hydraulic transient pressure analysis using preliminary hydropower equipment characteristics and plant operating data. As an alternate to the hydraulic transient analysis, a conservative pressure rise at the turbine of 10% to 20% can be assumed for Pelton type units; 30% to 40% for Francis type units without pressure-reducing valves and 10% to 20% with pressure-reducing valves. The pressure line then would vary linearly from the turbine to the first free water surface.

The designer must consider at least two types of suitable steel (low, medium, or high strength). High-strength steel may be required by design where the installation of heavy sections would be difficult (necessitating lighter sections) or where shop and field welding of thick sections might present problems. The designer should evaluate the potential for penstock corrosion and compare costs of the various methods available for corrosion protection. The cost of coatings and corrosion protection systems should be compared to the cost of added steel shell material.

Economic diameters for each of the penstock types must be calculated for the given energy costs and economic criteria, as well as the costs for the selected steel material and probable construction costs. Several formula can be used to determine the economic diameter. Flow velocities and regulation characteristics of the system must be checked to determine whether they are reasonable and conform to for similar installations. The required minimum thickness of steel pipe for handling must be compared to the thickness required to resist internal pressure and external loads. The greater thickness value should control design.

Thermal Stress: The conditions under which thermal stresses occur can be distinguished in two ways:

i) The temperature and shell conditions are such that there would be no stresses due to temperature except for the constraint from external forces or restraints. In this case, the stresses are calculated by determining the shape and dimensions the shell body would take if unrestrained and then finding the forces required to bring it back to its restrained shape and dimension. Having determined these "restoring" forces, the stresses in the shell are calculated using applicable formula.

ii) The form of the body and thermal conditions are such that stresses are produced in the absence of external constraints, solely because of the incompatibility of the natural expansions and contractions of the different parts of the body.

The thermal stress is very important parameter for support design.

Types of Penstock Support

A penstock can be supported in a variety of ways, depending on the initial design selected, existing geological conditions, and penstock profile. The penstock can be totally buried, partially buried, or supported aboveground.

Generally, a penstock is totally buried only when either access or drainage is required over the penstock, when the penstock requires protection, such as from falling trees, or when dictated by economics. The soil cover protects the penstock against the effects of temperature variations, as no expansion joints are required. A partially buried penstock can be very economical if the slope is stable and no more than 30 degrees. However, temperature loads on the penstock must be considered. For both totally and partially buried penstock, corrosion due to contact with the soil can be a problem and must be looked at carefully. Additionally, penstocks that are buried are difficult to inspect or repair. An aboveground support system is the most commonly encountered installation and allows a better handing of inspection and repair access and corrosion problems. However, the design must take into consideration the accumulation of longitudinal loads at the supports, and expansion joints usually are required.

Locations of Anchors and Piers

Penstocks installed aboveground normally are supported using piers with a spacing of 6 to 30 meter. The spacing depends on the diameter of the penstock, the subsurface bearing strength at the support, the type of end supports when not continuous, allowable coupling deflection, and the constructability of the penstock at the site. It is preferable to span as far as practical, but, the cost of the additional steel to handle bending stresses and concentrated loads at supports may be excess to the cost of installing piers. These two expenses must be balanced to obtain the most economical penstock; Piers are designed to support the dead height of the pipe and contained water, and resist the longitudinal forces resulting from temperature change, friction, and circumferential stress. Also, lateral loads from wind or earthquake may need to be considered.

Geological Conditions

Movement of the foundation at the piers and anchor blocks is not desirable and can cause excessive vibration and penstock overstress. Sound stable rock provides the best foundation for anchor blocks and piers, and if economically feasible, the penstock alignment should be located at or near surface rock during the initial layout phase. If sound rock cannot be found or reached for the foundations, and “softer” earth materials (e.g., weak or weathered rock and soil) comprise the foundation, settlements must be calculated and appropriate provisions made in the penstock design. Provisions for ground improvement, e.g. piles, may be necessary to attain suitable foundation conditions.

The surface and subsurface geologic conditions at the pier and anchor block sites should be investigated thoroughly by the experienced geologists to verify whether the natural foundation satisfies the design requirements or that foundation improvements are necessary to obtain satisfactory pier and anchor block installations. Weathering profile can be very erratic and must be carefully evaluated. Misjudgments about the weathering profile can result over-or under excavations for the foundations and may necessitate redesign of the supports. The bottom of the foundation should be below the frost line. The geology should be clearly described and portrayed in the construction document drawings to provide the contractor with all available information about the geologic characteristics of the foundation.

Foundation Stability

For sliding and overturning stability analysis, the factors of safety shown below are recommended minimums. The foundations material (soil, rock) resistance values used in this analysis be those determined by a qualified geotechnical engineer to be appropriate and allowable for design use for the specific site. Higher factors of safety are required when difficult foundation condition is encountered or when limited subsurface investigation has been performed.

Sliding: Following are values for the minimum factor of safety against sliding for a particular condition:

- Static pressure in-service condition: 1.5
- Dynamic conditions, transient pressures resulting from load rejection: 1.3
- Seismic loading (DBC)* combined with static pressure or test pressure: 1.15 - 1.25

Geologic investigations should be made prior to finalizing the value of the sliding coefficient of friction. These investigations also should be used in determining shear values of rock used for shear keys, bonding of rock anchors, and values to be used for passive resistance.

* *Design Based Criteria*

When shear keys are used, considerable care must be taken during excavation to ensure that the rock is not weakened by fracturing the rock. Once final rock excavation is complete, the rock surface should be sealed with either a gunite cover or thin concrete seal slab to prevent contamination. Prior to sealing, loose material should be removed by either an air or water jet.

When using passive soil resistance, the designer must recognize that movement of the soil is required to develop passive resistance. If this movement cannot be tolerated, then at-rest soil resistance must be used.

Overturning: The factors of safety against overturning for a particular condition are the same as for sliding. The resistance against overturning is a function of vertical loads, passive resistance if applicable, and rock anchor if used. The limitations on the use of passive resistance noted above in the sliding case also apply here. The bearing stresses should be calculated and the bearing capacity of the foundation material should be verified by geological investigation.

Anchor Blocks

The anchors are usually arranged where a vertical or horizontal change in direction of the pipeline occurs. However, if there is an exceptionally long straight length, intermediate anchor blocks are necessary at every 150-200 meter. An expansion joint is required between each pair of anchors. If the slope of the ground is appreciable it is usual, in order to assist in erection, to place the expansion joint at the top of a section of the pipeline. The section is then held at the bottom by the anchor and is free to expand upwards to the expansion joint. A schematic sketch of typical anchor loading is shown in Fig. 3.34. For basic principle for design of anchor block refer Annex-3.

Purpose: The purpose of anchor blocks is to fix the pipe line in place during installation and operation. They resist various forces acting on the pipe or bends and form straight pipe sections if there are no expansion joints. Penstocks supported aboveground, which are welded or have expansion joints or sleeve type couplings, require anchors at all points of changes in slope and/or alignment and sometimes at intermediate points in long tangents. Where expansion joints are used, a spacing of 150 meter between anchors and expansion joints is generally used because of the accumulation of longitudinal forces and the desirability of more fixed points during erection. Designs with longer spacing for anchor blocks do exist.

Loads: To determine the loads acting on the anchors, the following checklist should be used and break the loads are to be broken down into their vertical and horizontal components:

- Weight of anchor, pipe, and water
- Hydrostatic loads plus water hammer at ends of pipe
- Momentum loads caused by change of direction flow
- Thermal friction loads at ends of pipe (if mechanical couplings are used).

Design: Because the movement of an anchor could endanger the entire penstock and powerhouse, great care must be taken in its design. Coefficients of sliding and allowable soil pressures must be conservative. Each anchor site should be carefully examined, making borings or test pits where there is doubt. Anchors must be located on undisturbed material and must be protected against surface water which might undermine the structure or soften bearing material.

After the forces of the anchor have been calculated and reduced to vertical and horizontal components the anchor must be designed to satisfy the following conditions of stability:

- Sliding
- Rotation (overturning)
- Soil pressure

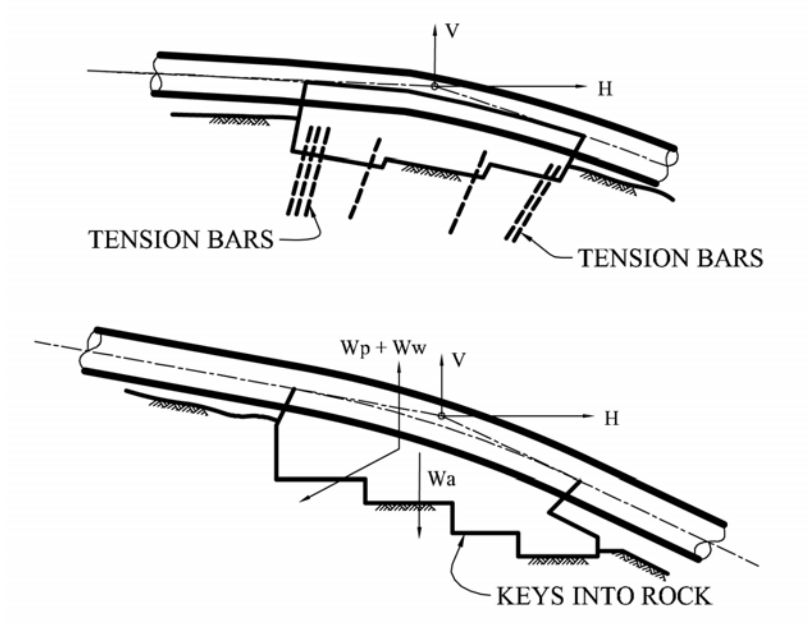


Figure 3.34: Typical Anchor Loading

Interpretation of symbols in figure 3.34 is presented below:

H = Horizontal component of the forces
V = Vertical component of the forces
W_a = Weight of anchor
W_p = Weight of pipe
W_w = Weight of water
+ sign when V is downward
- sign when V is causing uplift

a. Sliding

Stability against sliding is developed by shear- friction if the anchor is in good rock, otherwise, the design must be based on frictional resistance. Where the anchor is on rock and the vertical angle is small the horizontal force may be taken by surface roughness in the rock foundation. For safety against uplift the weight of the anchor must be at least 150 percent of the sum of the upward forces.

Where solid and massive rock is encountered, the unbalanced forces may be resisted by vertical and horizontal grouted rock anchor. The downward force of a rockbolt can be used to replace the volume or weight of concrete although the minimum volume of concrete must give a factor of safety of at least 1.5.

b. Rotation (overturning)

For stability against overturning the designer depends on allowable soil pressures that vary from site to site. For example, as much as 150 t/m^2 (15 kN/m^2) has been used for solid rock and as little as 30 t/m^2 (3 kN/m^2) for undisturbed firm soil. The resultant force must be located within the base and the allowable foundation stresses not exceeded. The resultant force should be located within the middle third of the base for two types of rotation: rotation in the downhill direction and rotation in the transverse direction (when there is a horizontal bend). If however, the rotation is in uphill direction, the resultant should be kept anywhere in the middle third of the base as long passive earth pressure on the uphill vertical face is within allowable.

c. Soil Pressure

Careful consideration must be given to the allowable soil pressure at each site.

Stress Analysis: The stress analysis for supported penstocks are to be carried out for the following:

a) Stresses between Supports

- Longitudinal stresses caused by beam bending
- Longitudinal stress caused by longitudinal movement under temperature and internal pressure
- Circumferential (hoop) stress caused by internal pressure
- Equivalent stress based on the Hencky-Misses theory of failure

b) Stresses at Supports

- Circumferential stresses in supporting ring girder caused by bending and pressure
- Circumferential stresses in supporting rings (if any) at saddle supports
- Longitudinal stresses in the shell at support caused by beam bending, and stresses in the shell caused by longitudinal movement of the shell under temperature changes and internal pressure
- Bending stresses imposed by the rigid ring girder; rim bending and circumferential
- Equivalent stress based on the Hencky-Misses theory of failure

c) Combined Stresses

Pipe thickness is often governed by combined stresses at locations of discontinuities. These locations, for example, are at anchors, supports, transitions, thrust rings, and stiffener rings. Sometimes, however, the thickness is governed by combined stresses where there are no discontinuities. An aboveground pipe section, as an example, might have maximum combined stresses midway between supports.

The resulting combined stress (known as the equivalent stress) is the stress that must be kept within the allowable stresses.

d) Tri-axial and Biaxial Stresses

The following is a general form equation for calculating tri-axial stresses:

$$2S_e^2 = (S_x - S_y)^2 + (S_y - S_z)^2 + (S_z - S_x)^2 \qquad \text{--------------------------------} \quad (3.101)$$

Where:
S_e = equivalent stress (N/m^2)

S_x, S_y, and S_z = principal stress (N/m^2)

+ = tension, and

- = compression.

Tri-axial stresses occur at a few places along a typical pipe section. They occur, as an example, at thrust rings and at ring girders. Biaxial stresses occur at many locations and in such case, S_z is usually zero. The term S_xS_y that appear in the formula for Biaxial stresses explains why shell thickness is often governed by a stress location where the principle stresses have opposite signs.

Pier and saddle support: The designer normally uses two types of supports for above ground pipelines-saddles for small diameter pipe and ring girders for large diameter pipe. There is not, however, a specific diameter that separate small pipes from larger pipes. Even the pipes of 3 meter in diameter have been found supported on saddles or saddles formed on high piers.

a. Pier

The piers are used to support the penstock between anchors must fulfil the following requirements:

- Provide pipe lengths convenient for shipping and erection
- Avoid excessive pipe stresses due to beam action between supports.
- Avoid severe local distortion of the pipe at the support
- Permit longitudinal movement but prevent lateral movement of the pipe.
- Maintain safe soil bearing pressures.

A spacing of 6 meter in lengths of pipe is convenient for shipping and erection in the context of Nepal. The pipe length should be evaluated for stress level, temperature, site conditions, and transportation.

For a high ratio of diameter to thickness, the local distortion around the saddle horns caused by circumferential bending may be unacceptable and require closer spacing of saddles or the use of stiffener rings.

b. Saddle

Saddles are a type of support for exposed penstocks, generally used for small diameter pipes and relatively small span. The support engages less than the full perimeter of the penstock, generally between 90 and 180 degrees of arc, and typically 120^0. Saddles are designed to support the weight of the pipe shell and water contained in the pipe, and to resist the axial friction force due to expansion or contraction of the pipe. The safety of the saddle should be studied against overturning, sliding & bearing capacity. An example of concrete saddle is shown in Fig. 3.35. For basic principle for design of saddle support refer Annex-3.

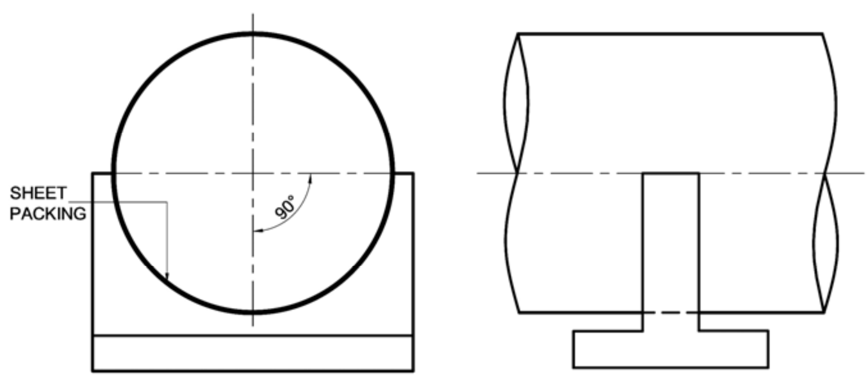


Figure 3.35: Concrete Saddle

c. Ring Girder:

Ring Girders generally are used to support long span exposed steel penstocks (Fig. 3.36). The purpose of the ring is to support the exposed penstock, its contents, and all live and dead loads. Also, ring girders stiffen the penstock shell and maintain the pipe section's roundness, thus allowing the penstock to be self supporting, acting as either a simple or continuous beam. The spacing of ring girders is governed by practical considerations. Shell diameter, thickness, and material type have a major influence on the spacing, which may range from 12m to 60m.

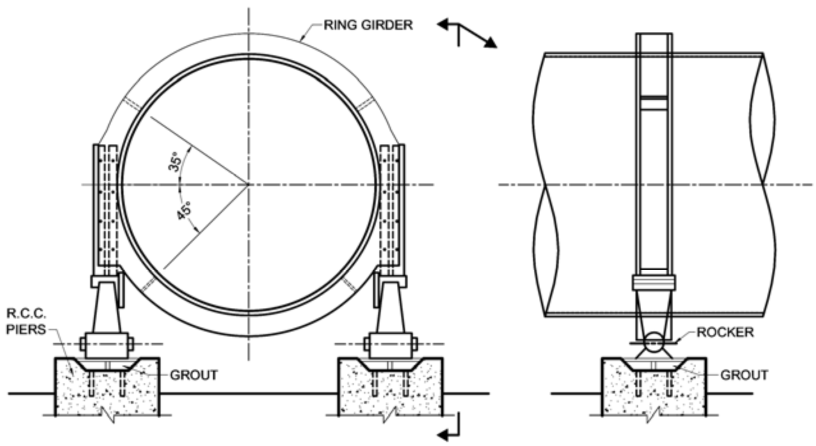


Figure 3.36: Ring Girder Support

Stiffeners: The stiffeners are provided on exposed penstocks when required to resist external pressure such as vacuum. The centre to centre spacing of stiffener rings should not be more than 240 times and not less than 60 times the thickness of the steel pipe.

Expansion Joints: Expansion Joints are installed in exposed penstocks between fixed point or anchors to permit longitudinal expansion, or contraction when changes in temperature occur and to permit slight rotation when conduits pass through two structures where differential settlement or deflection is anticipated. The expansion joints are located in between two anchor blocks generally downstream of uphill anchor block. This facilitates easy erection of pipes on slopes.

Expansion Joints should have sufficient strength and water tightness and should be constructed so as to satisfactorily perform their function against longitudinal expansion and contraction. Depending on the internal pressure, diameter of pipe and magnitude of movement expected, the following types of expansion joints are used for penstocks:

a) Sleeve type expansion joint, and
b) Bellows type expansion joint.

The expansion of the pipeline can be calculated as follows:

$$\Delta L = \alpha \times \Delta T \times L \qquad \text{------------------------------------} \qquad (3.102)$$

Where,

L = Length of the pipe section, (m)
ΔL = Change in length due to expansion (m)
α = Coefficient of linear expansion of steel
= 12×10^{-6} m/m ^{o}C
ΔT = Change in temperature (^{o}C)

To be safe, it would be recommended that the expansion joint can be capable of accommodating a length change of double this amount.

As for example a photograph of penstock of Puwa Khola showing expansion joint is given in Fig. 3.37.

Bulkheads:
Bulkheads are required for the purpose of hydrostatic pressure testing of individual bends, after fabrication of sections or whole of steel penstock and expansion joints, before commissioning. Bulkheads are also provided whenever the penstocks are to be closed for temporary periods, as in phased construction.

Figure 3.37: Penstock of Puwa Khola HPS showing Expansion Joint

Air Vents and Air Valves:

Air Vents and Air Valves are provided on the immediate downstream side of the control gate or valve to facilitate connection with atmosphere. Air inlets serve the purpose of admitting air into the pipes when the control gate or valve is closed and the penstock is drained, thus avoiding collapse of the pipe due to vacuum excessive negative pressure. Similarly, when the penstock is being filled up, these vents allow proper escape of air from the pipes.

The factor governing the size of the vents are length, diameter, thickness, and head of water, and discharge in the penstock and strength of the penstock under external pressure.

The size of the air vent may be determined by the following equation:

$$F = \frac{Q\sqrt{S}}{750000C}\left(\frac{d}{t}\right)^{3/2} \qquad \text{-------------------------------------} \qquad (3.103)$$

Where,

F = area of the air inlet, m^2

Q = flow of air through inlet, m^3/s

S = factor of safety against collapse of pipe

C = co-efficient of discharge through air vent (0.6)

d = diameter of pipe, mm

t = thickness of pipe, mm

Manholes:

Manholes (see for reference Fig. 3.38) are provided in the course of penstock length to provide access to the pipe interior for inspection, maintenance and repair. The manhole, in general, consists of a circular nozzle head, or wall at the opening of the pipe, with a cover plate fitted to it by bolts. Sealing gaskets are provided between nozzle head and cover plate to prevent leakage. The nozzle head cover plates and bolts should be designed to withstand the internal water pressure head in the penstock at the position of the manhole.

▲ INSPECTION OF PENSTOCK PIPE MANHOLE

Figure 3.38: Penstock of Puwa Khola HPS showing Manhole (source: NHE)

Branch Pipe: For hydroelectric facilities that have more than one unit, branching of the pipe line is required to divide the penstock flow. The penstock branching is defined as bifurcation when feeding two units, trifurcation when feeding three units and manifold when feeding a greater number of units by successive bifurcations. Branch pipes of bifurcating type are generally known as "Wye" pieces which may be symmetrical or asymmetrical.

Depending on the number of turbines it is frequently necessary to arrange for the pipeline to branch into two or three branches. Similar considerations apply both to bifurcations and trifurcations. The cross-sectional area of the water channels should be arranged so that there is no undue variation in the velocity through the trifurcation. It is inevitable, on account of the special shape necessary, that all the forces due to the water pressure are not automatically resisted by the shell plate as in the case of a circular section. It is therefore necessary to arrange a vertical diaphragm for transmitting these forces from the top to the bottom plates of the trifurcation. Alternatively, where it is not permissible to adopt a diaphragm, special heavy stiffening rings will be permissible in place of the diaphragm. Special heavy stiffening will be required on the outside in order to ensure that the trifurcation obtains its correct shape when the water pressure is applied.

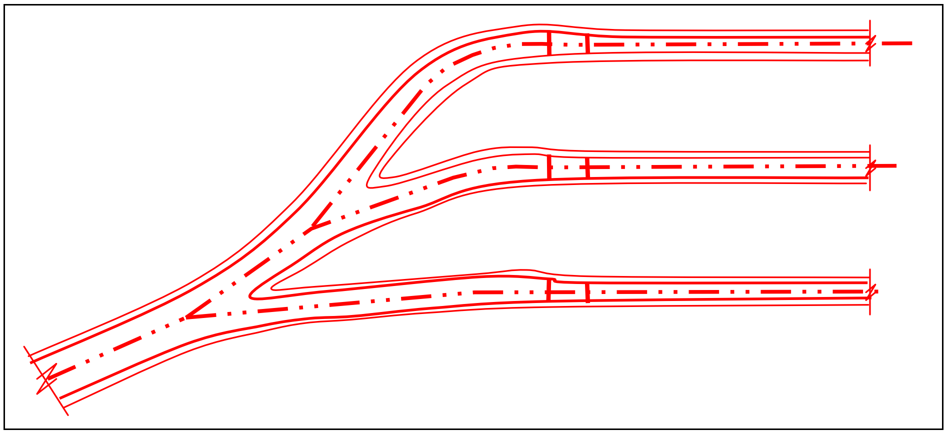


Figure 3.39: Schematic Sketch of Wye Branch Proportion

For hydroelectric facilities that have multiple units, Wye branches (Ref. Fig. 3.39) or bifurcations (Ref. Fig. 3.40) are installed to split or divide the tunnel or penstock flow.

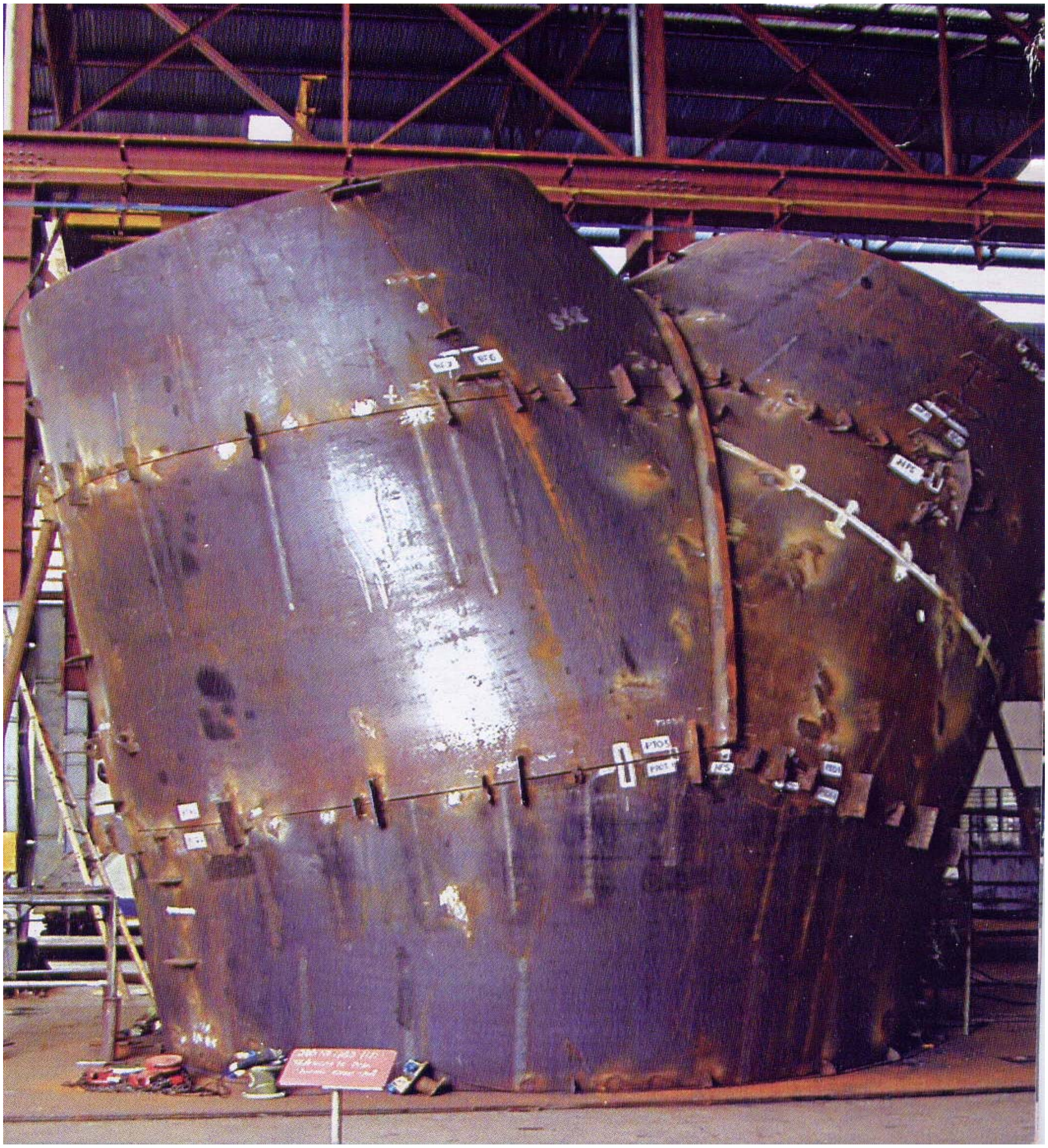


Figure 3.40: Bifurcation under Fabrication for Middle Marsyangdi HEP (Source: NHE)

For underground facilities, Wye branches are usually constructed of reinforced concrete cast against a solid rock mass where the internal loading is transferred to the rock. For outdoor powerhouses with surface penstocks Wye branches are usually fabricated of steel and are encased in a concrete anchor block to transfer the hydraulic thrusts to the surrounding foundation. Only steel Wye branches are discussed in this section. When the penstocks are steel lined it is preferable to locate the Wye-branches so that they can be designed in concrete. Wye should be suitably reinforced so that no substantial stress concentration or deformation occurs.

There are two major categories of bifurcating geometries: the straight symmetrical Wye (Fig. 3.41) and the manifold non-symmetrical Wye (Fig. 3.42).

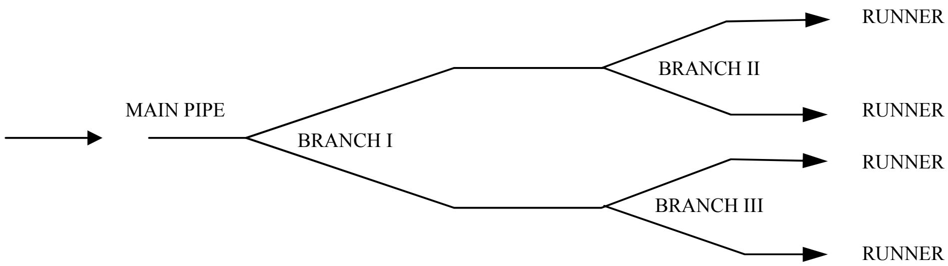


Figure 3.41: Straight Symmetrical Wyes

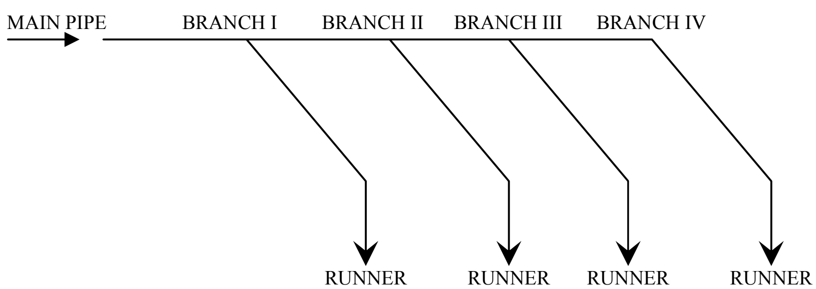


Figure 3.42: Non-Symmetrical Wye

The symmetrical Wye type may be single symmetric bifurcation or a series or bifurcating pipes in which the branch pipes are parallel to the direction to the main pipe. Generally, when the bifurcating pipe is the straight symmetrical Wye type, the internal angle between the two branching pipes should range between 60^0 and 90^0.

Non-symmetrical wyes distribute several branch pipes in the same direction from the straight main pipe as shown in Fig. 3.42.

To reduce the head loss, less bifurcating angle is advantageous. The less the bifurcating angle, the more reinforcing material is required at the bifurcating point.

Hydraulics*:* Wye branches should be designed for smooth hydraulic flow to avoid excessive head losses, vibration and cavitations. It must be geometrically detailed to evenly proportion the flow distribution to eliminate acceleration or deceleration of flow in the adjoining branches and thus minimise head loss.

Wye branches also must be detailed to direct the flow from the approaching Wye branch leg to match the design flow capacity of the adjoining branches.

In case of manifold non-symmetrical bifurcation, it is advisable to use a conical-shape design for the entrance into the branching pipes, rather than a true cylindrical shape, because the head loss using the conical pipe is about 1/3 of cylindrical pipe.

With respect to the head loss, the manifold type is exposed to a chance of having different head losses for each of the branch pipes, while the combination of single or multiple symmetrical bifurcating pipes has less such chances. In either case, particular attention has to be paid to ward off vortex formation and to make the construction insusceptible to vibration.

In addition to the bifurcation configuration, it is possible to design trifurcate Wye branches to be trifurcations in lieu of bifurcations. This design makes the construction more complex. Although the head loss is negligible at the centre pipe, the pipes on each side suffer a substantial reduction causing uneven head loss.

Several different branches should be detailed, initial costs estimated and the value of energy due to head loss calculated. The least total cost Wye branch should be evaluated in light of hydraulic adequacy.

Buried Penstock

Buried penstocks are supported continuously on the ground or soil at the bottom of a trench backfilled after pipe is installed. The adaptation of buried or exposed pipes is totally site specific. The exposed pipe is the more favorable option unless the local site conditions make the use of buried pipe advisable. The advantages and disadvantages of exposed and buried pipes are presented below:

Type of Penstock	**Exposed Penstock**	**Buried Penstock**
Advantages	• It provides more room for construction. • More accessible for inspection, maintenance and repair. This is predominating feature favorable to exposed pipe • Exposed pipe is usually less expensive to install	• The soil cover protects the penstock against effects of temperature variations. • It protects the conveyed water against freezing • It does not spoil the landscape • Immunity against slides, avalanches and falling trees • Concealment offers increased protection against damage caused by violence • Owing to the continuous support, this solution is also preferable from the structural point of view.
Disadvantages	• Temperature variation • The water conveyed may freeze • Owing to the spacing of supports and anchorages significant longitudinal stresses may develop especially in pipes of large diameter designed for low internal pressures.	• Such pipes are less accessible for inspection and maintenance • In case of large diameters and in rocky soil their installation is expensive • On steep hill sides, especially if the friction coefficient of the soil is low, such pipes may show a tendency to slide

When a steel penstock installation requires the pipeline to be buried below ground or within fill material, the penstock shell must be analyzed to resist not only internal pressure and other hydrodynamic loads, but also external loads due to earth fill, loads resulting from excessive deflections, potential movements of the ground, and numerous live loads. At least 1.0 to 1.2 meter covers should be provided for buried pipes.

Pipe shell design for internal pressure and other hydraulic and hydrodynamic conditions is done in the same manner as for the exposed penstock. The shell design for external soil and other loads must take into consideration the flexibility of the pipe and the shell thickness needed to resist the external loads acting on the pipe prior to pressurization.

External dead and live loads on flexible pipe produce compressive stresses in the pipe wall, which tend to reduce the hoop stress due to internal pressure. The key elements for shell design are the flexibility of the pipe and the type of soil and its characteristics.

Load Types: Two types of earth loads are applicable to buried pipe design;

- Trench loading, in which the pipe is laid in an excavated trench and backfilled
- Fill earth loading, in which the pipe is laid on a graded or prepared ground surface and fill is placed around and over the pipe.

External Pressure Design for Buried Penstocks: Buried penstocks must be analyzed for buckling due to external soil pressures and possible live loads on the backfill surface when the penstock is dewatered. The resistance of the pipe to buckling is affected by the density of the backfill material and the surrounding in-situ soil. Resistance to buckling can be improved by increasing the density of the surrounding soil. If the penstock can be periodically submerged, the analysis should consider buckling due to external water pressure and additionally check that uplift forces will not displace the dewatered penstock. If live loads on backfill dictate the thickness of the penstock shell, a reinforced encasement should be considered for vehicle crossings. The relation presented below represents the critical buckling pressure:

$$P_k = \frac{2E}{1-\upsilon^2}\left(\frac{t}{D}\right)^3 \quad \text{---} \quad (3.104)$$

Where,

P_k = Critical buckling external pressure for pipe shell in kg/cm^2

E = Young's modulus in kg/cm^2, 21 x 10^{6}kg/cm^2

υ = Poisson's ration, 0.3

t = Plate thickness of pipe shell in cm

D = Outside diameter of pipe in cm

Special Considerations: For a buried or an underground penstock installation, following points must be considered:

- Protection against corrosion by coating
- Cathodic protection
- Vacuum design
- Floating and drainage
- Welding and construction requirements
- Inspection, hydro-testing, and maintenance requirements

Stiffener Rings, Bends: These requirements of buried penstock must meet the same criteria as for stiffeners, bends in exposed penstock.

Steel Tunnel and shaft Liners

Steel liner is installed where:

a. It is required to control leakage out of the tunnel because of unfavourable geologic conditions;
b. There is insufficient rock cover to withstand the internal pressure within the tunnels such that a potential for undesirable leakage exists because of hydraulic jacking along horizontal or near horizontal joints;
c. Wherever the internal water pressure exceeds the minor principal stress in the surrounding rock mass such that potential for hydro fracturing or hydraulic jacking exists usually along vertical or near vertical joints if impermeable liner is not provided.

Steel tunnel and liners can be horizontal, vertical or inclined or shafts fulfilling the above three conditions. The project layout or study on different alternatives of such liners and construction time and methods etc. will govern the selection of vertical shafts or inclined shafts.

To properly assess criteria (c) above, it is necessary to know the magnitude and orientation of the in-situ principal stress in the rock mass at tunnel depth (tectonic plus gravitational effects). Unfavorable geological conditions would include, for example, the presence of highly fractured and shear zones or other possible highly permeable zones or discontinuities in the rock mass that intercept the tunnel and continue downstream to intercept access tunnels or adits or pass near the powerhouse cavern.

Several failures of unlined pressure tunnels and shaft have been attributed to hydraulic jacking. If necessary, in-situ tests should be used to determine the approximate maximum and minimum principal horizontal stresses at tunnel depth and their orientation.

If criteria (a), (b), and (c) above are satisfied, the steel liner may be terminated and an unreinforced concrete liner be provided beyond this point. This should preferably be confirmed by making water pressure tests in drill holes at the proposed cutoff site. The drill holes should be oriented to intercept the maximum possible number of joints. Water pressure should be equal to the maximum expected operating pressure.

Some judgment must be exercised in determining the minimum rock cover necessary to permit the rock surrounding the penstock to carry a portion of the internal pressure load through elastic interaction with the steel liner and concrete backfill. Where the rock cover over the penstock consists of several equivalent tunnel excavation diameters of massive, competent rock, it can be assumed that the rock carries part of the internal pressure load in accordance with the elastic interaction of the steel liner, concrete backfill, and surrounding rock, provided that the tension in the rock at a depth of 10 meters below the surface (based on the formula for the stress in a thick walled cylinder) is less than the compression at this point due to gravitation and Poisson's effects. This procedure was suggested by Jacobsen [1983] and is a logical approach.

Steel tunnel liners are normally required to prevent the migration of tunnel leakage due to unfavorable geological conditions in the surrounding rock mass. When a steel liner is installed in a tunnel, it must be analyzed to determine that it can resist internal operating pressures as well as resist buckling from external water pressures when the tunnel is dewatered infrequently for inspection and maintenance.

The analysis is carried out for the following:

- Internal Pressure
- External pressure

Design of Steel Tunnel Liners for Internal Pressure: The maximum internal pressure to which the steel tunnel liner can be subjected is discussed above. However the minimum required plate thickness can generally be determined considering load sharing with the contiguous rock surrounding the tunnel in elastic interaction when adequate rock cover and rock conditions exist. A transition between load sharing and no load sharing is considered near the underground powerhouse cavern. A distressed zone of jointed rock can exist adjacent to the cavern walls for several reasons. These include the effect of the trajectory of very high in-situ horizontal rock stresses passing over and under the cavern, unfavorable rock conditions gravitational effects, and loosening due to blasting during cavern excavation. The distressed rock zone will have a low modulus of deformation and as a result can share little of the internal pressure load with the steel tunnel liner.

External Pressure: The external design pressure for no watered penstocks should take into account the potential external pressure head that can develop on the steel liner from groundwater or high pressure water in the upstream power conduit migrating downstream through and around the grout curtain and seepage cutoff at the upstream end of the steel liner.

It should also account for grouting pressure. When the external groundwater pressure exceeds a head of 35m, the need for adopting drainage galleries over the penstock tunnels to limit external pressure head should be examined from a technical and economical standpoint, considering the geological conditions at the site.

External ring stiffeners should be considered when the external pressure analysis indicates that the critical external pressure requires an un-stiffened liner thickness greater than the thickness required for the internal design pressure. The final design should select a penstock steel such that there is a good balance in the plate thickness provided for internal and external pressures. Minimum handling thickness should not govern.

The critical external buckling pressure for an un-stiffened steel liner can be determined on the basis of Amstutz's formula [1970], considering an initial gap between the steel liner and the concrete backfill surround due to concrete shrinkage and temperature difference. (For Amstutz Bukling Pattern See Fig. 3.43)

External pressure design for no watered steel tunnel liners must take into account the potential external pressure head that can develop on the steel liner from various sources.

External pressure, the critical buckling external pressure is given by the following relation:

$$\frac{(1-V)^2.r_0.P_k}{E.t} = \frac{1-V^2}{(n^2-1)(1+\frac{n^2.L^2}{\pi^2 r_0{}^2})^2} + \frac{t^2}{12.r_0{}^2} x \left\{ (n^2-1) + \frac{2n^2-1-v}{1+\frac{n^2.L^2}{\pi^2 r_0{}^2}} \right\} \text{------------ (3.105)}$$

Where,

P_k = Critical buckling external pressure for pipe shell in kg/cm^2

L = Stiffeners spacing in cm

n = An integral number of buckling to make P_k at minimum

E = Young's modulus in kg/cm^2, 21 x 10^{6}kg/cm^2

V = Poisson's ratio, 0.3

t = Plate thickness of pipe shell in cm

r = Internal radius of pipe in cm

r_o = r + t in cm

<u>Amstutz Formulation:</u>

The critical external buckling pressure for an un-stiffened steel liner is determined considering a gap between the steel liner and the concrete backfill surround due to concrete shrinkage and a temperature difference.

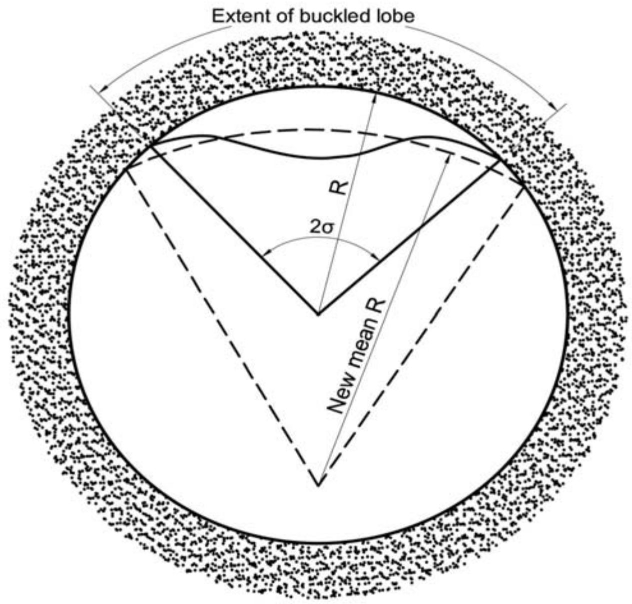


Figure 3.43: Amstutz Buckling Pattern

Amstutz developed his buckling theory (Ref. Fig. 3.43) for the forces and displacements on the pipe wall element represented by the mean arc line subtended by a corresponding new mean radius. The Amstutz formulas are presented in his paper "Buckling of Pressure-Shaft and Tunnel Linings."

The Amstutz formulas for determining the stress condition in un-stiffened cylindrical steel pipe are:

$$\frac{\sigma_N - \sigma_V}{\sigma^*_F - \sigma_N}\left[\left(\frac{r}{i}\right)\frac{\sqrt{\sigma_N}}{E^*}\right]^3 \cong 1.73\left(\frac{r}{e}\right)\left[1 - 0.225\left(\frac{r}{e}\right)\frac{\sigma^*_F - \sigma_N}{E^*}\right] \quad \text{----------------------} \quad (3.106)$$

$$p_{cr} \cong \left(\frac{F}{r}\right)\sigma_N\left[1 - 0.175\left(\frac{r}{e}\right)\frac{\sigma^*_F - \sigma_N}{E^*}\right]$$

Where:

$$\mathrm{i} = \frac{\mathrm{t}}{\sqrt{12}}, \mathrm{e} = \frac{\mathrm{t}}{2}, \mathrm{r} = \frac{\mathrm{D}}{2}, \mathrm{F} = \mathrm{t}$$

$$\sigma_{\mathrm{V}} = -\left(\frac{\mathrm{k}}{\mathrm{r}}\right)\mathrm{E}^*$$

k/r = gap ration, for ratio between steel and concrete γ (from table 3.44 & 3.45)
R = Tunnel liner radius
D = Tunnel liner diameter
t = plate thickness
E = modulus of elasticity
E^* = $E/(1-\nu^2)$
σ_F = yield strength
σ_N = circumferential axial stress in plate liner ring
μ = $1.5 - 0.5[1/(1+0.002E/\sigma_F)]^2$
σ^*_F = $\mu\sigma_F/\sqrt{1-\nu+\nu^2}$

v = Poisson's ratio = 0.3

P_{cr} = Critical buckling pressure

The values for different parameters for above equation for determining critical pressure can be taken from Amstutz curves presented in figures 3.44 and 3.45 below:

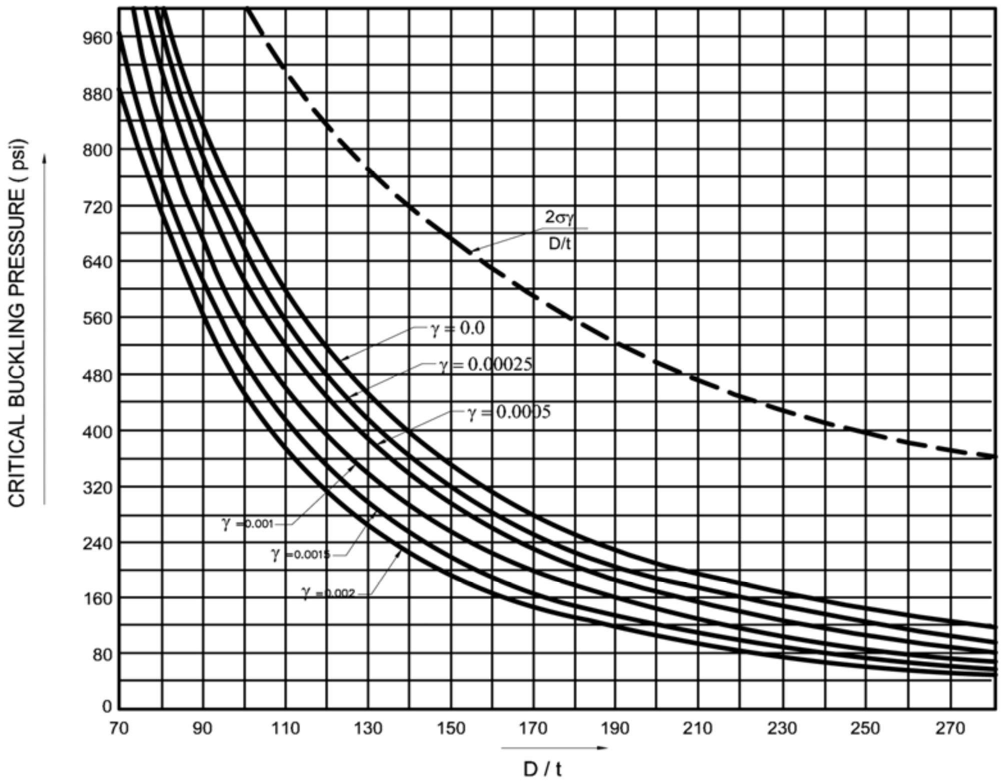


Figure 3.44: Amstutz Curves for Un-stiffened Liners (Yield Stress = 38,000 psi)

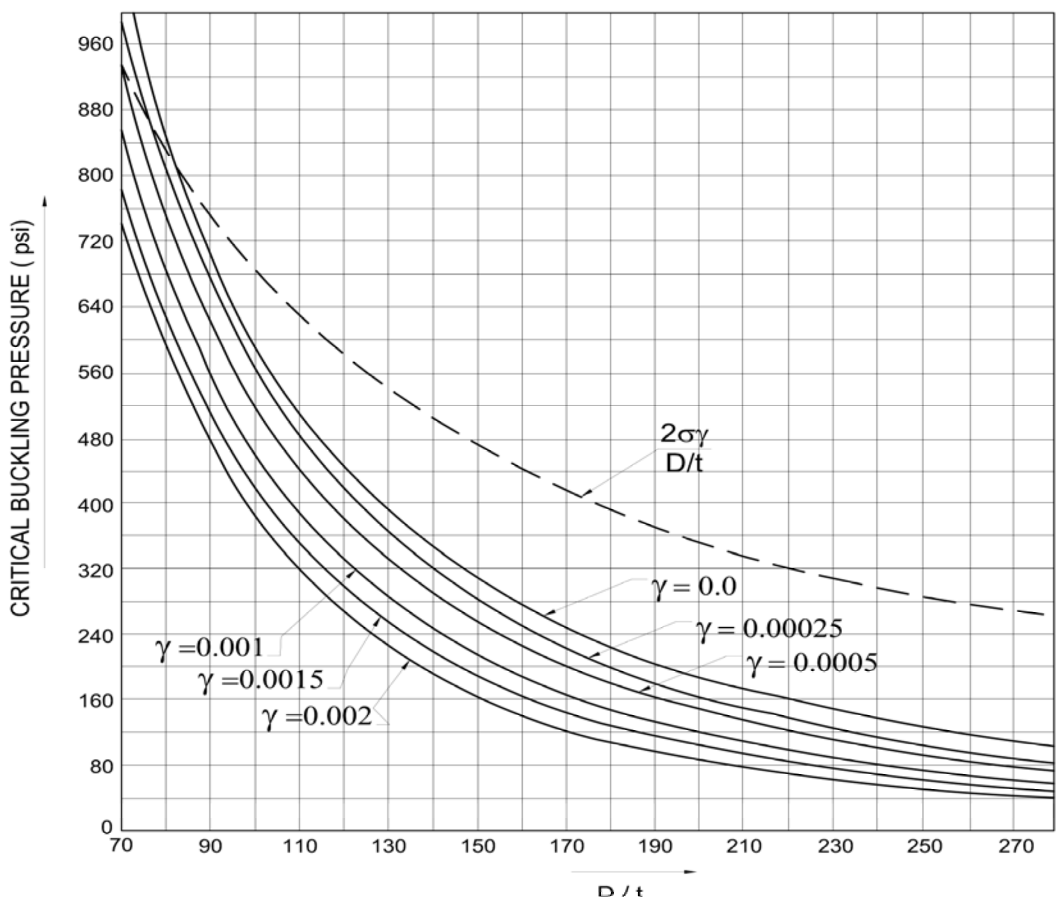


Figure 3.45: Amstutz Curves for Un-stiffened Liners (Yield Stress = 50,000 psi)

3.2.2.10 Manufacturing Standards and Rules

The following manufacturing standards and rules apply specially to penstocks and penstock parts that are fabricated by welding and constructed of carbon steel, low alloy steel, high alloy steel, or heat-treated steel.

Fabrication: The penstock can be fabricated in the manufacturer's shops or in a field fabricating plant in the project site, depending on the size of the shell. Fabrication of penstocks requires a variety of special machines and equipment, such as rolling machine, flame-cutting tools, and welding machines, testing, and handling equipment. The bigger size rolling machine which is available in Nepal with NHE can roll steel penstock having 3 meter in length and 42 mm in thickness (see Fig. 3.46).

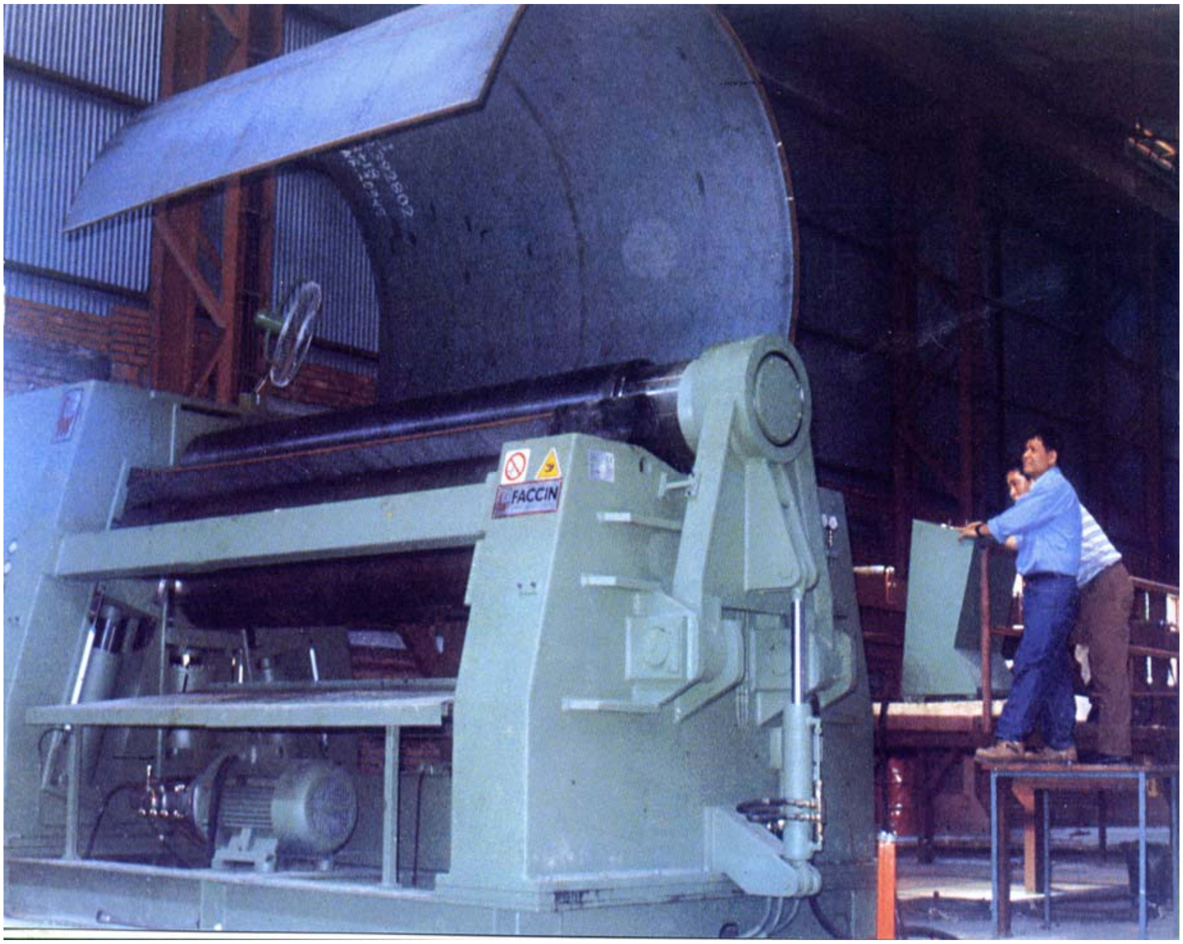


Figure 3.46: ROLLING MACHINE (Source: NHE)

Fabrication includes cutting the plates to exact dimensions, preheating the edges for welding, pressing and rolling the plates to the required radius, and welding the plates together. The type of edge preparation required depends on the welding procedure to be used. Shop joints are usually welded with semi automatic or automatic welding machines, while field joints are welded manually. In shop welding, after the plates have been rolled, they are tack-welded into pipe courses, and then full welding is done as per requirement. With the submerged arc-welding process, the joints are either machine or manually welded on both sides of the pipes. In the installation of a penstock in tunnel, there may not be sufficient clearance between penstock and tunnel walls to back-weld field girth joints from outside. In such cases, it will be necessary to complete the weld from the inside using an outside backing strip.

Stiffener rings may be formed from the plates or steel sections of suitable sizes. The segments so produced are then welded to the pipe with fillet welds either manually or by automatic machines.

Fabrication Standard: Fabrication of penstocks must confirm to the provisions of Section VIII, Division 1 of ASME code or equivalent. For detail, refer this code of practice or other equivalent.

The manufacturer must demonstrate that it has the shop facilities and also has had fabrication experience with specialty work of similar size and type within the past 5 years, as well as an internal quality control system similar to the SPFA (Steel Plate Fabricators Association, Inc., Westchester, IL) Plant Quality Certification Program.

Manufacturing Specifications: Fabrication of the penstock and penstock parts must be performed in accordance with the provisions of the ASME Code,' Section VIII, Division 1 or equivalent codes of practice.

Specifications for fabrication and installation of penstocks should provide for the requirements as discussed in the above and the following paragraphs. Specifications should be accompanied by drawings the general layout of the penstocks with sufficient design details to enable bidders to prepare estimates and shop drawings. In addition, the specifications should include fabrication, material, test, and erection requirements, and other essential information.

Materials: Materials used for steel penstocks and penstock parts must be furnished in strict accordance with the proper ASTM standard or equivalent and specifically in accordance with the requirements of ASTM or equivalent for pressure parts. Material for non-pressure parts, such as supports, thrust rings, lugs, and clips, need not conform to the specifications for the material to which they are attached or to the material specifications. However, if non-pressure parts are attached to the penstock by welding, they must be of weldable quality. The different type of steel used for penstocks fabrication for different hydropower projects in Nepal are high stress steel, general purpose structural steel and mild steel. The steel used for manufacturing penstocks for different projects have been purchased from different countries like Germany, Norway, Japan, China, India etc The different penstock parameters of Nepalese hydropower stations are presented in Table 3.16. The penstocks installed in Nepalese hydropower system are made from steel of different grades. Most of the penstocks installed are either surface type or embedded type. Only one drop shaft type penstock is installed in Andhi Khola Hydropower Station. Inclined shaft (embedded) type penstocks have been installed in many hydropower stations in Nepal like Kali Gandaki A, Marsyangdi, Kulekhani II, Chilime, Khimti, and Modikhola. The penstock installed in Kulekhani Hydropower Station is a combination of surface, buried and inclined shaft types.

Table 3.16: Penstock Installed in Nepal for different Hydropower station

S.N.	HP Stations	Installed Capacity (MW)	Type	Length (m)	Diameter (m)	Thickness (mm)	Rated head (m)
1	Kaligandaki 'A'	144	Embedded	243	5.25 – 2.8	21 -37	115
2	Marsyangdi	69	Embedded	75	5.0 – 2.8	N.A.	95
3	Kulekhani I	60	Embedded & surface	1324	2.1 – 1.5	9 -29	550
4	Kulekhani II	32	Embedded	843	2.1 – 1.5	12 - 25	284.1
5	Trisuli	24	Surface	71.66x4	2.3 -1.5	N.A.	54
6	Devighat	14.1	Surface	125	2.5	N.A.	
7	Sunkoshi	10.05	Surface		2.4	N.A.	30.5
8	Modikhola	14.8	embedded	314	3.5 – 3.2	15 -37	66.96
9	Panauti	2.4	Surface	370	1.4	N.A.	60
10	Seti	1.5	Surface	90	2.4	N.A.	22.5
11	Phewa	1.0	Surface		N.A.	N.A.	74.68
12	Sundarijal	.64	Surface	1700	.612	N.A.	228.65
13	Pharping	.5	Surface	533.5	.5	N.A.	207.9
14	Puwa Khola	6.2	Surface	1001	1.1 - .6	7 – 12	304
15	Khimti	60.0	Embedded	990	2.9 -1.8	12 -35	600
16	Chilime	20	Embedded	635.6	2.1 – 1.13	14 - 38	336.85
17	jhimruk	12.0	Surface	325	1.8 – 1.5	10 -13	205
18	Indrawati III	7.5	Surface	300	2.3 – 1.33	N.A.	60
19	Andhikhola	5.1	Vertical shaft	325	1.0 - .9	N.A.	250
20	Bhotekoshi	36	Surface & embedded	430	3.0 -	N.A.	
21	Chaku khola	1.5	Surface	180	1.0	8 – 12	80
22	Karnali Chisapani (proposed)	10800	Steel Liner	160 – 230 (18 nos.)	7.2	N.A.	165

Welding: Welding of shells and assemblies classified as pressure parts must be in accordance with the ASME Code, Section VIII, Division 1, or equivalent. No production welding can be undertaken until the welding procedures have been qualified. Only welders and welding operators who have been qualified in accordance with Section IX of the ASME Code or equivalent can be used in production.

Welding Process: Arc-welding and gas-welding process is restricted to shielded material arc welding (SMAW), flux cored arc welding (FCAW), submerged arc welding (SAW), gas metal arc welding (GMAW) and Gas tungsten arc welding (GTAW).

Welding Materials: Welding materials used for production must comply with the requirements of the ASME Code, Section VIII, Division 1, and Section IX or equivalent and the applicable welding procedure specification.

Edge Preparation: Shell plate edges must be prepared for welding as required by the applicable welding procedure specification. Edges of plates prepared for welding must be examined visually for signs of lamination, shearing cracks, and other imperfections. Defects must be removed by mechanical means or by thermal gouging processes.

Preheating: The minimum preheating temperature for all welded joints is 10°C (mandatory). An expert or welding engineer should be consulted about mandatory or recommended preheat requirements. The nonmandatory guidelines of the ASME Code, Section VIII, also should be reviewed. The procedure specification for the material being welded specifies the minimum preheating temperature under the weld procedure qualification requirements of Section IX of the ASME Code.

Postweld Heat Treatment: Weld procedures must conform to Section IX to the ASME Code or equivalent including conditions for postweld heat treatment. All penstock sections and parts must be given a postweld heat treatment at temperatures not less than those specified in the ASME Code, Section VIII Division 1 or equivalent.

Field Joint Ends: Ends of supplied penstock sections must be of the type specified by the purchaser. ANSI /AWWA C200-2 describe the following types to ends:

a. Plain-end pipe

b. Bevelled ends for field butt welding

c. Ends fitted with flanges

d. Ends fitted with butt straps for field welding.

Tolerances: The manufacturing and site installation tolerances for the penstocks must meet the requirements of the ASME code Section VIII, Division 1 or equivalent with regard to plus or minus deviation from true circular form.

Inspection and Testing

General

All penstock components are subject to inspection at the place of fabrication and/or installation. The inspector must be allowed to view any or all of the operations and have access to all report forms and radiographs.

Submittals

Upon request, the following documentation must be submitted to the owner:

(1) Fabrication, welding, and inspection procedures

Prior to the start of welding, copies of proposed welding procedure specification (WPS), procedure qualification reports (PQR), welding sequences, repair procedures, and welding rod control must be submitted. Documentation must include a weld map or table identifying each type of weld joint, the assigned welding procedure specifications, the parts being joined, the material thickness, and any requirements for preheat and postweld heat treatment.

Welding procedures must be qualified in accordance with Section IX of the ASME Code or equivalent. All procedures for weld and non-destructive examination (NDE) must be submitted prior to the work.

(2) Personnel qualification records

Welders and welding operators must be qualified in accordance with Section IX of the ASME Code or equivalent. Requirements for qualification include welder/welding operator performance qualification (WPQ) tests, recorded on ASME Code forms QW 482-484, 1 or their equivalents.

Non-destructive examination (NDE) personnel must be qualified under ASNT or equivalent (visual examination accepted). Qualification records of Non-destructive Testing (NDE) personnel must be submitted to the inspector upon request.

All non-destructive testing technicians and operators must be qualified at NDT Level II as defined in American Society of Non-destructive Testing (ASNT) or equivalent.

All inspection work must be performed by certified welding inspectors (CWI) who are certified in accordance with American Welding Society (AWS) QC13 provisions.

(3) Weld examination and inspection reports

These reports must include detailed records showing evidence of quality of welding. For each section of weld inspected, a report form is required. The report must identify the work and show the welder’s identification, the area of inspection, the acceptance of the welds. For more detail, refer the code of practice mentioned above.

Coating and Lining Inspection Requirements

When corrosion protection, internal lining, or external surface coating is specified in the purchase order, a careful inspection for strict compliance with the specification, referenced standards is mandatory.

Final Shop Inspection

- Ensure that all required test reports, as-built drawings and other documentation generated during the fabrication of the penstock are available and processed in accordance with the procurement documents.
- Prior to shipment, perform final inspection on the penstock sections, and verify that all deficits have been corrected and that each section has been properly marked.
- Verity the identification marks on each penstock section for conformance with specification and drawing requirements. Confirm that the top arc bottom centre lines of special sections have been marked and are clearly visible on both ends of each section.
- Verify that each penstock section has been properly braced to prevent damage during transit and handling. Obtain assurance that the penstock sections will be correctly blocked for shipment and that coated penstock sections are loaded on padded bunks or saddles as required.

Final Field Inspection

The entire penstock, including all coatings and linings, must be inspected for damage that may have occurred during shipment and erection. All damage must be repaired.

Damage Correction

Damaged areas, such as scratches, gouges, grooves, or dents as determined by the inspector, must be corrected as specified in the project specification.

Field Weld inspection

All field welds must be inspected in conformance to the project specification and to the requirements of Section VIII, Division 1, of the ASME Code or equivalent.

Field Inspection of Linings and Coatings

All coatings and linings including those applied in the shop or field must be examined for damage that may have occurred during shipment and erection according to the appropriate requirements of the specification referenced standards. All damage must be repaired.

Cathodic protection systems must be tested for electrical continuity. If an impressed current system is in place, the power source must be given appropriate performance tests. Documentation in the owner's file must detail operating procedures.

Reports

Ensure that all required test reports, as-built drawings and other documentation generated during fabrication and erection of the penstock are available and processed in accordance with the procurement documents and specification.

Copies of reports for field and shop welding NDE and repair procedures must be included in the documentation in the project file.

Non-Destructive Examination

General

The type of non-destructive examination (NDE) for all butt joints and certain full fillet lap joints in pressure retaining parts is established by the designer in conjunction with the weld joint reduction factors.

The design engineer must document the required non-destructive examinations for these weld joints in a Fabrication and installation NDE data sheet. Non-destructive examination for acceptance of any material subject to hydrostatic pressure testing must be performed prior to the hydrostatic tests. Reported defects must be retested by the same NDE methods used for the original tests. If the engineer determines that the specified non-destructive testing is not possible because of conditions encountered in the work the engineer must choose another method of inspection, the engineer may require additional tests and examinations.

Non-Destructive Inspection of Welds

After completion, welded joints are usually radio graphed to detect defects in welds. Weld defects may consist of slag inclusions, cracks, gas pockets, porosity, incomplete fusion, and undercutting. Cracks, incomplete fusion, and undercuts are not acceptable but a certain amount of porosity, slag inclusions, and cavities may be acceptable if their size and distribution is such as not to impair the strength of the weld. Criteria for judging the acceptability of defects are given in the ASME code or other equivalent. Apart from radiograph, the following non-destructive inspections of the welds are recommended for different service conditions:

- Ultrasonic Testing (UT) as per ASME RECOMMENDATION.
- Magnetic Particle Inspection as per ASME RECOMMENDATION.
- Dye Penetrate Testing etc. as per ASME RECOMMENDATION.

Radiographic Examination

All radiographic examination must be in accordance with ASTM or the ASME Code section V, Article 2 or equivalent. Double film must be used.

Film must be marked with the date, owner's specification number, contract number, piece or section number and weld number procedures must include an identification system that ensure traceability between the radiographic film and the weld examined. Radiographs and interpretation reports must be submitted to the owner, upon the request for permanent retention.

Similarly, the technical details of Ultrasonic Test, Magnetic Particle Test, Liquid Penetrant Test and Visual Examination can be referred from ASME or other equivalent code of practice.

Installation

Upon delivery of penstock sections at the specified delivery point, they are usually transported to the place of installation by truck or trailer and lifted in place by cableway, derrick, or other means. For installation in tunnels, special handling equipment consisting of trolley and hoists is required. After being set in line and grade on temporary supports, several pipe sections are first tack-welded together, and then joints are completed by welding. The completed welding joints are inspected by non-destructive testing either by radiograph or by Ultrasonic testing as recommended by the applicable Standards or Code of Practice.

Corrosion

To protect the penstock against thinning by corrosion, erosion, mechanical abrasion, or other environmental effects, it shall be provided with a protective coating. Some fixed value is added up to the base metal to compensate such effects, known as corrosion allowances.

Bonded dielectric coatings protect the substrate from corrosion by forming and electrically insulating barrier between the substrate and the electrolyte. The effectiveness of bonded dielectric coatings is a function of the bond developed between the coating and the substrate, the impermeability of the coating, and its continuity and durability. Although the coating may perfectly continuous at the application plant, defects will occur during handling, shipping, and installation, and in service after installation. The defects usually are in the form of pinholes damaged areas.

Surfaces beneath bonded dielectric coatings cannot serve as anodic or cathodic sites. Depending on the degree of continuity (efficiency) of the coating, the reactive surface area can be greatly reduced. The reduction of available surface to serve as cathodic sites greatly reduces the cathode-to-anode surface area ratio, resulting in a concomitant reduction in corrosion activity.

The bonded dielectric coatings specified for steel penstock typically exhibit efficiencies of 99% and greater. That is, only 1% or less of the surface area exposed to behave anodically or cathodically. This in turn reduces the corrosion activity as discussed previously.

Protective Linings and Coatings

Corrosive environments to which penstocks are exposed dictate the use of high-performance, industrial coatings and linings in conjunction with premium surface preparation, application, and cure.

Protective Linings

Protective linings are materials used in the interior of penstocks to protect the metal from corrosion and wear.
Linings may be applied in the field at the time of installation or in the shop after fabrication. It is always necessary to field repair the coating after installation.

Some materials are temperature dependent; coal tar enamel has withstood 25 meter per second with water temperatures below 10°C. Coal tar epoxy may withstand 12.5 meter per second at temperatures below 10°C. Some lining materials may cause cavitations problems if they have a high degree of friction.

Protective Coatings

Coatings for the exterior surfaces of the penstocks protect against corrosion and provide a pleasing appearance. The coatings may be applied completely in the shop at the time of fabrication or in the field at the time of installation. Certain coatings should be applied in the shop where there are proper facilities. Coatings in contact with potable water also must meet Environmental Protection Agency (EPA) health requirements.

Protective coatings are the most commonly used methods for corrosive protection. Improper surface preparation and application can lead to premature failures of the coatings. The interior of penstocks are almost always coated with either coal tar enamel or coal tar epoxy and have proved to provide excellent protection at a reasonable cost.

Surface Preparation

The penstock surface must be prepared in accordance with the standard referred to in the specification for the material selected or the manufacturer's instructions if no recognized standard is specified. The following instructions should be followed for the preparation of the painting surfaces:

- Surfaces to be encased in concrete must be cleaned and coated before concreting
- During or after abrasive blasting, any accumulations of slag, dirt, blisters, weld spatter, metal laminations, or other irregularities must be removed by appropriate mechanical means. Rough edges and welds must be smoothed or rounded and then brush coated.
- Bolts, nuts, and other appurtenances to remain uncoated must be cleaned and masked prior to penstock coating
- Piezometer and flow meter holes must be plugged prior to the coating application.
- Metalwork to be welded must be primed with a weldable inorganic zinc prior to shipping.
- Metalwork must be coated after welding.

Coating Application and Cure

Coating materials must be applied in accordance with the standard referred to in the specification of the material selected or the manufacturer's instructions if no recognized standard is specified. This includes, but is not limited to, materials mixing, application, dry film thickness, handling, and pot-life, if applicable. Limitations on ambient and substrate temperatures, relative humidity, and dew point must be observed as stated in the reference standard or in the recommendations of the manufacture of the coating material. The contractor must be experienced in the application of the type of coating specified. In addition, the following general procedures must be followed (some procedures do not apply to all coatings)

- Until the coating is dry or cured the penstock must be free of airborne dust. Exterior coating must not be applied during winds in excess of 30 kmph unless enclosures are provided. Surfaces that become contaminated between applications of coats be re-cleaned prior to application of the next coat. All surfaces to be coated or lined must have temperatures 3^0 C above the dew point.
- A coating system agreed upon by the owner and the contractor must be used for the bolts, nuts and other appurtenances.
- The curing time between coats must be rigidly followed.
- Items that are normally factory coated may be furnished primed or primed and top coated provided such coatings are compatible with subsequent coatings to be furnished separately.
- A liter sample representing the coating material must be furnished at the time of shipment for each type, bath, lot and color of liquid and mastic used when quantities exceed 100 liters. The constituents of multiple component coatings must be furnished separately.
- When multiple coats are applied, successive coats must be shaded sufficiently to denote color differences between coats. The final coat color must meet the design specifications.
- Penstock sections must be identified to facilitate proper erection without marking the exterior of any exposed part of penstock coatings.
- Samples of coal tar enamel must consist of 10 kilograms of the enamel and 4 linear meters of fibrous glass.

Hydrostatic Testing

General

A proof hydrostatic test on the penstock after installation is most desirable. The need for hydrostatic testing is determined by the engineer. The following should be considered in determining the need for hydrostatic testing:

(1) Site location, automatic shut-off systems, and head

(2) If the site location poses high risk for loss of life or property, the testing may be desirable

(3) Structural complexity: complicated weldments, such as wye branches, compound and simple bends, high pressure section of straight pipe shell may warrant testing to verify the design and fabrication. The need for shop and field hydrostatic testing should be mentioned in the contract specification considering the points mentioned above.

Component Testing

Pipe for mitered bends may be hydrostatically tested prior to making mitered cuts. Girth welds in mitered bends that have not been hydrostatically tested must be tested for defects using 100% radiographic or ultrasonic examination.

Fittings and attachments, such as nozzles, ring girders, and anchor rings, must be welded to the shell prior to hydrostatic testing. By agreement, nozzles, manholes, ring girders, and anchor rings may be fabricated into previously hydrostatically tested pipe with all welds provided they are tested using the appropriate Non Destructive Examination (NDE) procedure.

Types of Hydrostatic Testing

If it is determined that hydrostatic testing is necessary, the location and time of testing must be established in the project specification. Hydrostatic testing can be performed either in the fabrication shop or in the field (after the penstock has been completely installed) or by a combination of both.

Shop Hydrostatic Testing: Testing must be performed prior to application of the coating and lining materials. Sections with attachments must be tested in the horizontal and upright position; test pressure must be measured at the centerline of the section. If hydrostatic testing machines are used, the end seals must not produce any significant inward pressure on the ends of the section.

The shop-tested sections must be filled slowly with water and pressure increased slowly until the required test pressure is reached. The test pressure must be held for 10 minutes, released and then reapplied immediately and held while the test section is observed. There must be no leakage. Any defects in the work must be repaired, and the section must be hydrostatically tested again.

The hydrostatic tester must be equipped with a calibrated recording gauge to record the test pressure and the duration of time that pressure is applied to each length of pipe. Records for charts must be available for examination at the plant.

Field Hydrostatic Testing: When required by the engineering or the specifications, the completed penstock must be hydrostatically tested in the field. For buried penstock, sleeves-type couplings must not be buried until completion of the hydrostatic test and/or watering-up. To facilitate construction or to avoid over-pressuring, the penstock may have to be tested in sections, using bulkheads for isolation.

For field tests, some loads imposed on the structure may not be the same as those during operating conditions. The installation must be checked, prior to testing, to determine that anchorages and bulkheads can withstand the selected test pressure. Vacuum valves or standpipes must be installed to prevent vacuum collapse in the event of failure of a portion of the penstock under test.

The test pressure must be held long enough to ensure that the entire section under test can be inspected and any leakage detected and the leakage rate measured. The leakage rate is determined by metering the water required to maintain pressure. Depending on the nature of the installation, the time required to perform the hydrostatic test may vary considerably. However, the duration of testing must be sufficient to ensure the detection of any problems. This duration may range from 1 to 24 hours. Upon reaching test pressure, readings from pressure gauges must be recorded at 10 to 30 minute intervals depending upon the duration of the hydrostatic test. Continuous surveillance for leakage must be performed during the first 30 minutes, and the test section must be visually inspected for leakage at 1- hour intervals thereafter. The hydrostatic must be performed during daylight hours if practical. If any portion of the test is performed during night hours, suitable lighting for inspection must be provided.

Hydrostatic test are accepted on the basis of the leakage rates described in the project specification. If a break or unacceptable leakage occurs during any of the testing operations, the test must be terminated and the engineer notified.

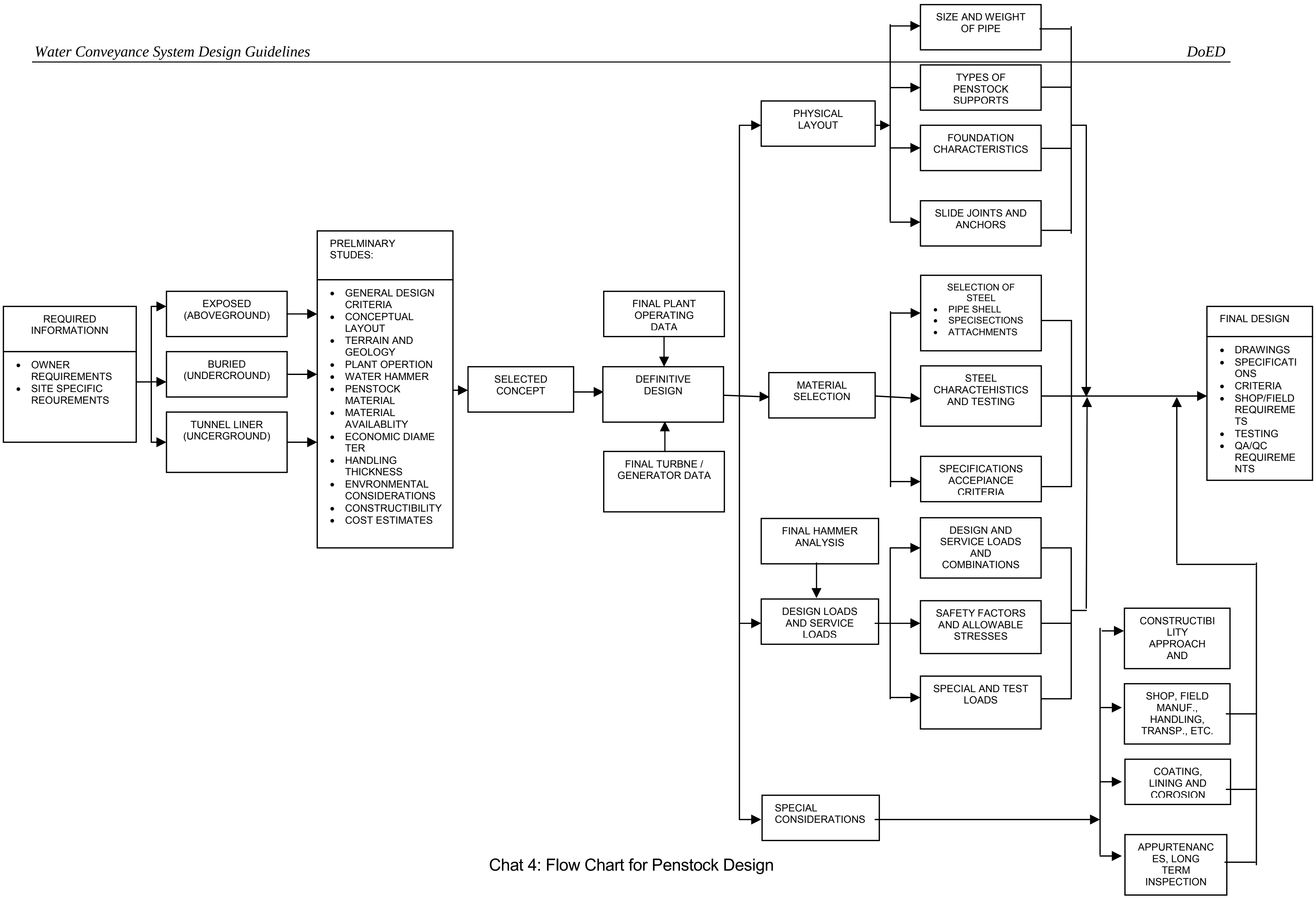
Defects disclosed during the hydrostatic test must be repaired. The section must be hydrostatically retested.

Safety Factor

For the steel material of penstocks a factor of safety of from 3.0 to 3.5 based on the tensile strength, is usually adopted, thus the allowable stress σ for a material having an ultimate tensile strength of 3,700 kg/cm^2 is not higher than 1,200 kg/cm^2. The allowable stress is usually reduced for elbows and wyes (a factor of safety of 5 to 6 is used).

3.2.2.11 General Flow Diagram for Penstock Design

The general flow diagram for penstock design is presented in chart 4 below:



Chat 4: Flow Chart for Penstock Design

CHAPTER – IV

GUIDELINES FOR CONSTRUCTION

4.0 GUIDELINES FOR CONSTRUCTION

4.1 Introduction

Successful Construction of water conveyance system of hydropower projects are conceived, planned, designed and built by a project team consisting of an owner, design professional and constructor. Within this context, quality is achieved when each team member competently and, in a timely fashion, fulfills his / her obligation in cooperation with other members. The purpose of the guidelines is to:

- provide recommendations for owners, design team, and constructors on how to provide quality construction;
- clarify and define the roles, responsibilities and limits of authority for each participant;
- offer general and specific definitions of critical words and phrases;
- provide detailed specifications with technical, general, special and miscellaneous requirements together with the standard provisions.

4.2 Quality of Construction

Characterization of quality construction are as follows:

- Meeting the requirements of the owner to functional adequacy, completion on time and within budget, life-cycle costs, and operation and maintenance.
- Meeting the requirements of the design professionals as to provision of well-defined scope of work, budget to assemble and use of qualified, trained and experienced staff; budget to obtain adequate field information prior to design; provisions for timely decisions by owners and design professionals; and contract to perform necessary work at a fair fee with adequate time allowances.
- Meeting the requirements of the constructor as to provision of contract plans, specifications and other documents prepared in sufficient detail to permit the constructor to prepare priced proposals or competitive bids; timely decisions by the owner and design team on authorization and processing of change orders; fair and timely interpretation of contract requirements from field design and inspection staff; and contract for performance of work on a reasonable schedule.
- Meeting the requirements of regulatory agencies (the public) as to public safety and health; environmental considerations; conformance with applicable laws, regulations, codes and policies.

Quality in the construction is also characterized by complete and open communication among all project parties; selection of qualified organizations and personnel by the owner for all phases of the project; change orders in publicly bid projects within a predetermined budget; rapid resolution of conflicts and disagreements; and absence of litigation.

The construction is generally carried out by a team consisting of an owner, design professional and constructor. Other participants play a more limited role. The major participants should be involved in nearly all phases of construction. The owner as the originator is responsible for leading and directing the team. The owner selects the other team members by set procedures discussed elsewhere.

4.3 The Owner's Role

The broad responsibilities of the owner include two major factors:

a. Develop a complete and realistic requirements and objectives;

b. Provide a thorough understanding to other team members of the role and responsibilities of the owner.

A relationship where the design professional is working with the owner as an advisor on all aspects affecting design is beneficial to both the designer and owner, and is recommended. It allows the design professionals to suggest various alternatives, order of magnitude cost ranges, and tradeoffs in other related aspects.

An owner cannot expect poorly communicated objectives to be fulfilled. The owner and design professional should develop good lines of communication and agree on how the objectives will be achieved and what expectations are reasonable. Through discussion of all relevant facts, recordings, and delineation of such exchanges will greatly enhance the probability that expectations will be adjusted and objectives fulfilled.

The owner should understand important concepts and practices, such as life-cycle cost, peer review, alternatives studies, value engineering, construction contract documents and shop drawings. The owner should be cooperative, plan for proper communications, and insist that construction adheres to project requirements.

4.4 Communication and Coordination

Clearly communicating information such as requirement, expectations, scope, costs, schedules, and technical data is a vital element. A coordinated effort among various team members is required to achieve an integrated program. Insufficient coordination and communication have heavily contributed to failures and problems and to the dissatisfaction of team members. The frequency of lawsuits has been highest from clients in Nepal with whom communications are difficult and those with limited construction experience.

The owner, design professional and constructor are all equally responsible for proper communication during design and construction.

4.5 The Project Team

Hydropower projects are conceived, planned, designed and built by a team consisting of an owner, design professional, and constructor. Each team member has to fulfill competently and, in timely manner, his obligation in cooperation with the other members. Each team member has to have functionally selected experts assisting in construction activities. The organization arrangements integrating the roles of the owner, design professional, and contractor may be the traditional one where the owner contracts independently with the other two parties. In other arrangements, the owner may issue only one contract to design / construct firm [Engineering Procurements Contract (EPC), Built Own Operate and Transfer (BOOT), Built, Own and Operate (BOO), etc], or the owner may perform his own staff, employing an outside design professional or constructor. Discussions hereunder emphasize the traditional arrangement and outline the contractual requirements necessary to define this arrangement.

It is the responsibility of the owner to administer his contracts with the other team members and to monitor and coordinate the activities of all parties involved in the planning, design and construction. The owner may discharge his responsibilities more effectively by retaining a project manager authorized to act for the owner.

In addition to the specific responsibilities listed, other responsibilities apply equally to all team members. These consist of accepting responsibility, striving for economy and efficiency, cooperating and coordinating with other team members adhering to the established budget, schedule and program and insisting on quality.

4.6 Procedures for Selecting the Design Professionals

Selecting the proper design professional is critical for the construction. No two professional design organizations have the same training, experience, capability, or culture. It is necessary for the owner to carefully structure and administer a selection procedure which secures a proper fit between the abilities of the design professional and the construction requirements.

This design professional is committed to competition on the basis of project-specific qualifications. This commitment to selection on the basis of demonstrated professional competence is to be reinforced by legislation. Selection on the basis of price bidding is viewed as counter-productive. The recommended selection procedure requires to submit statements of interest and qualifications in response to the owner’s invitation and statement of requirements. These responses are evaluated by the owner according to previously announced selection criteria. After the design professional is selected on the basis of qualifications, contract negotiations between the owner and design professional are initiated. During these negotiations, definition of scope of work, schedule, compensation for design services, and other contractual matters are agreed upon and documented in a written contract.

It is important that both the owner and design professional begin the design phase with attitudes requiring excellence in performance, rather than lowest possible design costs. Project failures can be expected if minimum design cost is the primary basis for selecting the design professional. The best agreement results from establishing a fee after extensive scoping discussions occur, which utilize the experience and knowledge of the design professional, owner and advisors to the owner.

4.7 The Agreement for Design Services

For the best interests of both the owner and design professional and for the successful completion of a quality construction, it is imperative that the design professional and owner have a clear understanding, an agreement in writing, of the duties and responsibilities of each party. Without such an agreement, incorrect assumptions or misunderstanding may develop, jeopardizing the mutual trust and confidence. If standard contract forms with refined variations to meet owner’s specific concerns are used, and specific detailed agreements are prepared covering the extent of each party’s duties, responsibilities and authority.

4.8 Peer Review

Peer reviews are recommended as added safeguards for the public, the owner, and the design professional. A fresh, unbiased and diplomatic review by an independent, high-level professional can be a highly cost-effective measure. The overall time to complete a project can be reduced by a peer review.

4.9 Planning for Construction

Planning is necessary to meet the objectives, including quality in construction. In planning for construction, providing adequate resources to construct a quality work and verify conformance with requirements is essential. When planning in construction operations is effective, adequate lead time to mobilize certain critical resources is secured and the responsibility for quality performance is clearly assigned.

Planning for quality construction requires an accurate assessment of the owner’s capabilities and level of involvement or quality control assistance is required from outside sources. Proper planning includes implementing quality requirements in contracts and purchase orders. Construction planning includes recognizing the need and requiring the involvement of key representatives of other organization at the side. This full scope of planning for construction is necessary to fulfill requirements for projects.

The owner’s key roles are to form the project team as early as possible, assign responsibilities and establish levels of performance, include qualifications for quality performance as a part of bid evaluation, establish the contracting and purchasing program, and plan for necessary site representation from each project team member.

The design professional's major responsibility in planning for quality construction is fully to define quality requirements. The constructor and his representatives should prepare plans and procedures, plan to provide the resources necessary for construction, and make construction contributions to key decisions by the owner and the design professionals. Modern techniques of planning are to be used including the network with the Critical Path Method (CPM), Program Evaluation and Review Technique (PERT) and others.

4.10 The Construction Team

The primary goal of the construction team is to build a quality project within budget on time and with little or no litigation. There are several different contractual arrangements which define the construction team. They include the traditional owner, design professional and constructor arrangement and a variety of construction team. Members of the team are affected differently depending upon the system used.

All members of a quality oriented construction team have a serious interest in team performance. Some members, however, are not directly involved in daily operations. Financial organizations, insurance companies and surety representatives, utilities, supplies, government officials, attorneys, and the ultimate users are, to different degrees, interested in progress and play some role in producing the final project. Owners, of course, are key members of every construction team and are responsible for decisions and actions to maintain progresses and minimize delays or other distractions. Delays often are used as a basis for seeking redress in the courts.

4.11 Competitive Bidding Procedures for Selecting the Constructor

Successful use of competitive bidding requires clear and complete contract documents. An obvious collateral benefit of such documents is a better informed construction work force. Use of appropriate responsibility criteria or prequalification procedures will help to provide a key element of a quality project a capable constructor. Furthermore, by applying fundamental competitive principles in selecting the constructor, the owner will greatly enhance the probability that he will not pay an exorbitant amount for constructor services.

Competitive bidding is the most widely used method of selecting constructors. It is also mandated by law, bidding is subject to various rules and procedures. If bidding procedures are clear and detailed, all team members in the construction process will benefit. Constructors will be motivated to bid if the rules are fair and clearly outlined. Furthermore, the bidding phase will be organized and less prone to dispute, which in turn allows everyone to focus on the objectives of entering into a fair contract.

Many competitive bidding procedures are contemplated or specified in different documents and reproduce here is beyond the scope of this study.

4.12 The Construction Contract

A Construction Contract is the agreement of two or more parties regarding the construction of a defined project. There is insufficient data available to indicate that any particular set of documents is the standard or commonly used document throughout construction industry. Each agency in Nepal has its own variety of documents.

It is important to mention here that the then His Majesty's Government of Nepal (HMG/N), in exercise of power conferred by the Rule 62 of the Financial Administration Regulation, 2056 (1999) has framed the Public Works Directives and made them mandatory for all the engineering works including construction and procurement of goods and services. These directives are in the following four parts:

Part I : Organizational Directives

Part II : Procedural Directives

- Volume A General Procedures
- Volume B Sector Procedures

Part III : Standard Procurement Documents
- Volume A Works
- Volume B Goods
- Volume C Consultants

Part IV : Compendium of Reference Documents

First edition of these documents printed in January 2002 is in the market and should be the guiding documents for all the practicing engineering professionals, constructors and the owners. Issues which are not covered by these documents are to be taken from the international practices prescribed by the owner.

Construction contract participants should be aware of several common problems relating to contract documents that are discussed in other paragraphs of this chapter.

4.13 Planning and Managing Construction Activities Contract

Essential elements of planning and managing construction include clear communications through planned reporting scheduled meetings, memos, shop drawings processing, and review of progress payment requests. Project management tools include formulating and regularly updating the construction plan and schedule, estimates and the quality control program. There are many ways to plan and manage construction activities. Depending upon the options selected roles and responsibilities of team members may vary. But following responsibilities do not alter:

- The owner is responsible for activity coordination, contract enforcement, and stopping work (except in emergencies);
- The design professional is responsible for design changes and interpretation of contract documents.
- The constructor is responsible for construction methods, direction of labor and job safety.

Many other elements of construction activities require careful consideration including site facilities and services, safety and first-aid (including public safety), loss prevention, project clean-up and public relations. Final close-out should involve joint participation of the owner, design professional and constructor.

4.14 Contract Administration Procedures for Construction

There must be at least two parties to every construction contract and each party has contract administration responsibilities. This discussion is presented from the perspective of the owner's representative. The representative is either the owner's employee or a consultant retained by the owner who is responsible for monitoring cost control, quality control and progress reports of the construction.

Most owners have different expectations of administrative requirements. However, there are certain professional mandates, management principles and communication imperatives that apply to contract administration procedures. These are:

- Quality Commitments;
- Payments and Cost Control;
- Progress reports;
- Timely and acceptable receipt of contract deliverables;
- Liaison requirements;
- Communication skills; and
- Record keeping and retrieval functions;

Contract administration can be envisioned as a three-dimensional matrix. Along the primary axis are the three major monitoring functions – quality control, cost control, and progress reports – all of which must be performed by the contract administrator. Along another axis is a time line dividing those functions into preconstruction, during construction and interests always involved in every construction phases. The third axis represents the three owner, constructor and design professional. The effective administrator must balance the rights and requirements of all three.

4.15 Use of Computers

With proper training and an awareness of its power and limitations, a sophisticated computer system or program can be a valuable tool to the constructor. Specific computer programs can perform many time-consuming administrative functions quickly and efficiently, thus making the constructor more efficient. Computers allow construction personnel to accurately and rapidly monitor changes in contract commitments, costs, schedules, impact of change (variation) orders, sub-contractor relationships, material deliveries and many other aspects. As the job progress, profit and loss can be monitored. Resource productivity can be greatly improved and potential problems can be identified. Organizational efficiency can be fine-tuned, job estimating can be made more accurate and cost of sequence work curtailed. Computers can also improve communications and teamwork within the project team by allowing access to project information and allowing the project team to function with greater efficiency. Use of computers should be made mandatory.

4.16 Shop Drawings

The contract document should specify the responsibilities and authority of the parties involved in the production and processing of shop drawings, design professional has prime responsibility to specify shop-drawing requirements and procedures and owner has to review it. But the contractor has to provide shop-drawings and submit the drawings on schedule and design team has to make timely review and approval. Also the contractor has to timely provide techniques of construction and construction safety.

The review of shop-drawings by the design professional does not relieve other parties of their contractual responsibilities or their responsibilities under registration law.

4.17 Specifications

The term “Specifications” as it relates to construction means a detailed written description of the owner’s design concepts. This written description, together with the tender drawings, has as its primary function the communication of the design concepts to the constructor. Specifications perform an additional function of great importance from an engineering point of view. They have also an administrative – legal function. The provisions of the specifications become conditions, in a contract law sense, affecting the legal rights and duties of the owner and the constructor. For purposes of clarity specification provisions are categorized as follows:

a. The Technical Requirements: Here one has to describe the owner’s technical requirements in achieving construction objectives. Also there should be provisions which describe the consultant materials of and the process to be used in producing concretes together with a description of the process to be employed in its placement.

b. The General Requirements: Under this division the administrative and legal condition under which work is to be performed has to be described.

c. Special Requirements: This division of the specifications consists of a group of provisions which are chiefly non-technical in nature but which are nevertheless tailored to particular project and therefore cannot conveniently be placed with the “general” or “standard” provisions.

d. Miscellaneous Requirements: Additional provisions and documents, which are not in the nature of specifications, may nevertheless be part of construction contract between the owner and contractor. Foremost among such additional provisions and of particular importance to competitive procurement of construction service are bidding provisions. Other contract documents to be included are the “Notice to Contractor” and others.

4.18 Project Start-up Program

Start-up is a transitional phase that focuses on preparing a facility for occupancy and use and testing the equipment and systems in that facility. Quality in this phase is achieved through the use of skilled personnel, adequate planning, suitable tools and procedures, proper definitions of job requirements and appropriate supervision and technical direction. Quality is verified through surveillance, inspection, testing, checking and review of work activities and documentation. Quality verification must be performed by individuals who are not directly responsible for performing the work activity.

4.19 Operations and Maintenance

Project team members cooperate to produce a project which can be reliably, safely, and efficiently operated and maintained. These important activities are the final steps in providing a quality construction. Emphasis is on including O & M consideration early in the project design. This has been discussed separately.

4.20 Risk, Liabilities and Conflict Avoidance

The risk of bodily injury, property damage, financial loss and legal liability is inherent in the construction process and mistakes can have serious consequences. Risk can be managed through quality of performance of work and careful selection of projects team members, terms and conditions of contracts.

There are key elements to this management progress. In the early stages, the nature of the project, history of participants and the appropriateness of the financial compensation of the parties are determining factor of risk. The construction contract outlines the risk assumption, risk avoidance, and risk transfer mechanism. Owners provide the resources for construction, design professionals supervise suitability of function, and constructors oversee compliance and control of risk of life and property. Society at large enforces its values and requirements through the interpretive process, i.e. codes, regulations and legal requirements. Risk transfer mechanisms, limitations, indemnities, warranties, and insurance assign and dispose of inordinate risk.

A quality-oriented construction process requires special effort. Quality-control procedures should be instituted as part of each constructor-subcontractor relationship. Such procedures may utilize checklists as a tool in risk assessment and conflict avoidance. The end result should be strict enforcement of functional requirements, codes, regulations, and construction quality.

CHAPTER – V

A GUIDE FOR OPERATION AND MAINTENANCE

5.0 A GUIDE FOR OPERATION AND MAINTENANCE

5.1 Introduction

Maintenance is essential for the safe and efficient running of all machines. The implementation of a scheme will involve a large capital commitment. In order to repay this investment the scheme needs to run efficiently and continuously through out its design life. Efficient and continuous running will only be possible with skilled operation of the scheme and a well planned maintenance programme.

Maintenance can be reduced to some extent by good design, but it will never be eliminated. Moreover, good design is not necessarily that which reduces maintenance to a minimum, but which contrives that the combined prime cost, operating, and maintenance charges are a minimum consistent with maximum safety.

Maintenance in itself must be efficient and this does not necessarily mean perfect. The maintenance of any hydro-electric installation in a state of perfection would quickly become almost as uneconomic as no maintenance at all; but maintenance in relation to safety must take precedence over economic considerations.

Maintenance must, therefore, be watched closely and must be carefully guided along its proper line, so that it is neither excessive nor slipshod. The greatest difficulty in justifying expenditure is that lack of maintenance may not be evident as the direct cause of a failure, or the ultimate expenditure resulting from lack of maintenance may not appear on the balance sheet for a considerable number of years, but when it does the resulting costs may be very large indeed, so that it is wise to err quite heavily on the side of excessive maintenance.

On a hydro-electric development of any size there should be a permanent maintenance squad, the size of which is determined by a relatively small number of regular maintenance jobs which require a certain minimum man-power. This minimum-sized squad will usually seem excessive for several months of the year, and every possible advantage should be taken during these months of doing all those maintenance jobs which appear to verge on the "excessive", but which will ultimately prove themselves to have been very well worth doing.

The line of demarcation between maintenance and repair is of necessity nebulous and flexible. It depends largely on the size of the works and the consequent size of the maintenance gang. It also depends of local contracting firms. In general, the work of the maintenance gang should consist of inspection, testing, minor repairs, and a few minor new works and improvements.

The maintenance gang should not, as a general rule, embark on any one undertaking that will keep them fully employed for more than two or three weeks, or it will be found that they are not readily available to perform their true function of maintenance. Large repair works may be no contractor to undertake such works, in which case the maintenance squads capable of working independently or together as the need arises.

In the following notes a number of the more obvious maintenance works related to the water conveyance system are mentioned, and remarks have been added to show how good design can reduce the work of maintenance squad. The work has been listed in the direction of flow of the water from the high-level interception works down to the river bed below the power station.

5.2 Responsibility

The design engineer, the equipment installers for different components of conveyance system and the users of the scheme all have important part to play in the operation and maintenance (O&M) of hydropower schemes.The designer must have O&M in mind throughout the design process. He or she must have an understanding of the skill levels, motivations, availability and costs of O&M staff in order that appropriate designs are used. For example, an unlined earth channel may be appropriate where access is good and full time maintenance staff is envisaged but totally inappropriate for an unattended plant in difficult terrain. The design engineer must write the O&M schedules. He or she must get to know the users of the scheme very well in order to make sure that the skills availability and motivation of O&M staff will commensurate with the schedules. The schedules must specify clearly which kinds of task should be undertaken by directly resident O&M staff, and which types of maintenance should be referred to the higher concerned authorities. Similarly the owner and the operator of the system must be well trained for the proper maintenance of the system.

5.3 Canals

Unlined open channels are only suitable for very small interceptions; they suffer from erosion of the banks, from subsidence of cuttings in sidelong ground, and from obstruction by floating debris and siltation.

Maintenance of such conveyance systems consists largely of keeping the channel clear of debris, patching the banks, and encouraging the growth of vegetation on the exposed slope of cuttings. The effects of drifting debris may be minimized by the judicious use of fences in vulnerable areas. Maintenance of the canal should include inspection of blockages. Any stones, silts or vegetation should be removed from the canal. It is important to repair any damage as soon it appears. If leaks are not attended to, they get worse and sometimes wash away the ground that holds the canal. Drainage to carry rain water away from canal should always be inspected and cleaned or repaired as necessary. Any repair and maintenance of the canal should not go beyond the original design of the canal itself.

5.4 Aqueducts

Both open and closed aqueducts are commonly used for the water conveyance system. The open aqueducts will have the same problem as it was in the canals. The comparatively high maintenance cost of open aqueducts, lined or unlined, has led in recent years to be adoption of piped aqueducts. Erosion and floating debris problems scarcely arise on piped aqueducts, but for inspection purposes fairly frequent manholes must be introduced together with catch-pits to intercept rolling stones and sand, which seem to find their way into any and every form of aqueduct.

The catch-pits require clearing out by the maintenance squad at least once a year, and the pipes should be swept through at the same time. The manholes, however, introduce a problem of their own. In conditions of severe forest there is often a little water in a piped aqueducts; this water will probably freeze, and with the thaw long strips of quite thick ice will start to float down the aqueduct; these often become wedged at a manhole, resulting in a blockage and overflow with consequent erosion around the manhole, which may endanger the foundations. The design of manholes is receiving special attention on the Scottish hydro-electric schemes, as is the provision of light baffles to guide floating ice.

The patching of concrete in pipes, pipe supports, and manholes will fall to the lot of the maintenances squad, who should also attend to the internal and external paintwork if any.

Piped aqueducts may be laid in trenches or above ground on stools; there is remarkably little difference in cost between the two methods. If the pipe is laid above ground on stools, care must be taken to ensure that local hill water does not undermine the stools, and, in cases where the pipes appear partly above ground, there must again be adequate drainage to prevent any ponding against the pipe. These points should be checked periodically by the maintenance squad.

5.5 Desanding Basin and Forebay

Desanding basin forms an important part of hydropower scheme as it is the silt tank that determines the wear on the turbines. It is therefore wise to keep the silt tank in good conditions at all the time. The collected silt should be removed as when necessary if not it will collect up to the limit and any excess will pass into the turbine. Operation of the upstream and downstream gates of the desanding basin, for the flushing of the deposited sediments, will have to follow the operation manual. Other than the proper silt flushing from the tank to prevent turbine weir, it required very little maintenance. The occasional repair of concrete, gates and flushing passage and other civil structures will have to be carried out in the suitable time.

Flushing valves/gates may need attention as there are moving parts which need to be lubricated about once a week. The forebay tank will contain the trash rack which will need daily maintenance of cleaning as it is here that all the water borne vegetation, floating debris is prevented from entering the penstock and turbine. The trash rack should be cleaned as often as possible. During rainy period it may be necessary to clean even twice a day. For the trash rack cleaning special trash rack cleaning device might have to be installed.

5.6 Hydraulic Equipment

The need for meticulous maintenance of the hydraulic equipment on conveyance system cannot be too heavily stressed. Turbines and electrical gear in power stations are usually very well maintained, but, regrettably, this is by no means always the case with the hydraulic equipment. Poor maintenance of the power station plant may lead to loss of revenue and high repair charges, but neglect of flood regulating equipment, tunnel gates, etc., may lead to a local or even a national disaster.

The real test of flood control equipment usually arises in a combination of adverse circumstances, each small in itself. A sudden thaw with heavy rain, roads still blocked by old snow drifts, a very minor fault in automatic gear, a wrong decision—and a chain reaction of small troubles is soon multiplied into disaster which might never have happened had the maintenance gang noticed that the heater element in a relay switchbox had been defective for a short period or that some small filter was not completely clean.

The maintenance gang and operating staff must be made completely familiar with every part of the hydraulic equipment, and should examine and operate all items of the equipment at regular intervals. It is often inconvenient from the power generation point of view to operate certain parts of the equipment, but this inconvenience must be faced in the interests of safety, and certain fixed periods should be arranged each season for the testing of all parts of the hydraulic equipment.

At each test period there should be a thorough examination followed by adjustment, cleaning, oiling, greasing, and painting as necessary, and a written record should be maintained giving exact details of the attention given to each item. It will often be found that a standard form of record sheet acts as a valuable reminder to the men engaged on the work. It is important that the correct grades of oil and grease should be used as recommended by the suppliers of the machinery and wherever there are leather seals or washers a vegetable oil such as castor oil or its commercial equivalent should be used.

Certain parts of the equipment are located under water, for instance gate guides, sills, lintels, and sometimes lintel seals in the form of staunching bars. These should be examined and cleaned whenever the opportunity arises, and care should be observed when repainting the gate guide castings to ensure that machined roller paths are kept completely clear of paint. Gates will readily become jammed in their guides due to the presence of a little paint mixed with some flue dust and sand.

When examining the underwater parts of the equipment, or gates that have been submerged for some time, a careful watch should be kept for the formation of rust nodules. These form readily in certain waters and may be detected by a slight bulging of the paint surface. On scraping, the paint over the nodules usually crumbles away readily revealing a speck of rust and some pitting on the metal surface beneath. Hard wire-brushing or scraping to shining metal, patched over with three coats of bituminous paint, is about the best repair known at present.

Important hydraulic equipment should, whenever possible, be housed in weatherproof buildings, which not only protect the machinery from the weather but afford protection to the maintenance men. In the course of their duty, and so promote good and efficient work. Such buildings can also be heated during inclement weather, with beneficial effects to the life and efficiency of the machinery. This space-heating should be quite independent of any permanent built-in heaters in electric motors and switchboxes.

During maintenance work, all seals and soft-metal sealing strips should be adequately protected. Nuts removed for maintenance purposes should be put back with a touch of grease on the thread and preferably be sealed with paint. During painting of metal work, adjacent concrete should be carefully covered as paint splashes on it are most unsightly and very difficult to remove.

The efficiency of certain types of hydraulic machinery may be impaired by the formation of ice. For example, gate seals and exposed gate roller tracks may easily become clogged. In some catchment areas a quick thaw may release flood waters before it releases the gates, and the maintenance gang should be fully alive to this danger.

5.7 Tunnels

During its early life, a tunnel should receive comparatively frequent inspection; say at the end of the first year's use and again after two more years. As a result of these inspections it should be possible to forecast the period that may reasonably be allowed to elapse until the next inspection, which period may well be five years in the case of a normal-sized tunnel under moderate pressure. Large-sized tunnels and those subjected to really high pressures should be inspected more frequently.

The draining, inspection, and refilling of the tunnel is normally organized by the station or area superintendent, but much of the detailed work and organization will fall to the maintenance squad. The maintenance foreman must be fully acquainted with the procedure and with details of the tune that should be taken over emptying and filling.

When emptying a tunnel, it must be borne in mind that natural ground water and water that has seeped out of the tunnel into the rock will tend to return to the tunnel; hence if pressure in the tunnel is allowed to fall too rapidly, external pressures may cause serious damage to tunnel linings, particularly to the comparatively flat inverts of horseshoe or D-shaped tunnels. These external pressure effects are even more important in the special case of steel-lined shafts, inclines, and tunnels, and are mentioned in more detail later.

A tunnel can usually be filled a little more quickly than it is emptied, but reasonable time should be given for stresses to distribute themselves and for temperatures to settle down and, on a long length of tunnel on an easy gradient, the rates of filling must be slow enough to ensure that there is no violent surging, resulting the trapping of air and roof slapping. It is quite easy to do serious damage to the tunnel roof by filling too quickly.

At the tunnel intake there must be an adequate air shaft downstream of the main gate. This shaft usually vents into the gatehouse which should have ample louvers and, during tunnel filling, all the doors and windows of the gatehouse should be properly opened, or they may easily be damaged by bursts of trapped air escaping from the tunnel.

If there is a steel pipeline at the downstream end of the tunnel, the early stages of filling should be carried out with the utmost care so that the pipeline is stressed gradually and is not subjected to violent temperature .changes. A by-pass valve at the tunnel intake, for use in filling the pipeline, is a useful adjunct, as it is very difficult to ensure that a large main gate is cracked open by the very small amount that is usually necessary.

The maintenance gang will play an important part in the tunnel inspection and should be equipped with powerful electric or paraffin pressure lamps giving a diffuse light and not a beam. A complete and accurate record should be made of all apparent damage to the tunnel lining; even small faults may increase very quickly and become serious, and any points where water is seeping or running into the tunnel should be noted. Clear chainage markers in the tunnel are of great assistance in locating defects, for comparison after each inspection.

If on inspection it is found that some remedial work is necessary, it is usual, unless the work is very obviously of a vital nature, to refill the tunnel and continue generation of power until all the necessary stores, tools, etc., have been got together, and until the area generation program has been adjusted to allow for a prolonged shut-down of the tunnel concerned. It is unwise to start on the repair with incomplete stores and equipment in the hope that the balance will be forthcoming. The planned approach leads to better and more economical work, and affords the opportunity of adding minor repairs to the major program, including the cleaning out of sumps, repainting of the main gate, etc.

Minor faults in concrete linings usually appear at the vertical joints of shutters, at the joints between side walls and inverts, and in any “cold joints “that may have occurred during placing of the lining.

The usual minor faults that occur in a tunnel lining can well be dealt with by the maintenance gang and largely consist of patching concrete. Patching by means of mortar is virtually useless; defective concrete should be cut away to an ample depth and replaced by concrete placed against shutters. Large patches should be reinforced with steel rods drilled and grouted into the sound surrounding concrete or rock. Any deterioration of the concrete lining that is obviously a purely surface deterioration may be patched with gunite provided the area is not great. It is advisable to protect all patchwork whether concrete or gunite with an ample coating of bituminous paint.

Excessive leakage into the tunnel may well indicate a point of leakage from the tunnel when it is full; such places should be dealt with by pressure grouting, on completion of which the grout holes should be carefully plugged with a fine concrete, not with a pure mortar.

While the tunnel is empty the opportunity should be taken of inspecting the main gate; if a bulkhead or emergency gate is provided upstream of it, this should be lowered and sealed, so that the upstream face of the main gate and its guides may be inspected and attended to as necessary.

Most mountain waters produce an organic growth of one type or another on the concrete lining of tunnels; this fine slimy weed-like substance will in the course of time become sufficiently thick to have a material effect on the hydraulic characteristics of the tunnel, and its roots will affect the concrete lining. These algae must therefore be cleared away periodically. If the tunnel can be held out for a considerable time, perhaps during some major repair, and can be sufficiently ventilated, these organic growths will dry out, when they may be hosed off the tunnel walls quite readily. The humid conditions which exist in most tunnels and the cost of laying out mains often preclude the use of this method, though it is the best, and resort must then be made to brushing with stiff brooms.

Bituminous paint is thought to have a deterrent effect on the rate of growth of algae in tunnels and on underwater concrete structures generally, but there is insufficient evidence to prove this point. It may be that the application of bituminous paint destroys the roots of old growth and so prolongs the period before new growth becomes excessive.

Immediately before refilling the tunnel, a methodical inspection must be made to ensure that no tools have been left behind. It is remarkable how easily a wheel barrow, shovel, or length of scaffold tube hides itself away in a dark tunnel. Great care must be taken to ensure that all men are out of the tunnel, and a strict system of working passes should be employed if any large number of men are to enter the tunnel on the day it is to be refilled. Any manholes or other entrances to the tunnel should be closed methodically as the inspection party proceeds up the tunnel.

When re-sealing manholes, new sealing rings should be employed, and in the bolted type the correct size of spanner should be used for the nuts without the addition of long pipes, crow-bars, etc., and the nuts should be drawn up in proper sequence in the manner of tightening down a motor-car cylinder head.

5.8 Steel-lined Tunnels

The use of steel linings in tunnels has become quite common in recent, years, especially with the increase in the number of underground power stations that have been built. This type of construction has led to the adoption of drop-shafts and high-pressure tunnels in place of exposed pressure pipelines leading to conventional stations. In these steel-lined tunnels the strength of the surrounding rock is taken into account in determining the necessary weight of steel, and considerable savings result, a valuable factor with the steel shortage which has existed.

Steel linings are comparatively thin and are susceptible to collapse by buckling if subject to high external pressures; special care is necessary when de-watering such tunnels.

At inspection time, the steel lining should be looked at with a view to discovering any signs of bulging; any sign of water entering the lining should he treated at once by drilling through it, grouting under pressure, and finally making good the lining by welding. During the grouting process, the steel lining must be carefully watched to ensure that it is not buckled by the grout. Steel lining should be tested with a tap hammer periodically to ensure that no voids have been developed between it and its concrete backing; any large areas of void should be grouted.

It is extremely difficult to prepare steel lining for painting once the tunnel has been in use, as the humid atmosphere will tend to keep it damp so that the paint will not adhere properly; the invert section is the most difficult to deal with. Sprayed metal coatings of zinc or aluminum, or a combination of the two, applied at the time of construction, afford the best-known protection and, in view of the extreme difficulty of ever re-painting linings, it is advisable to give them the best possible protection in the first instance.

5.9 Pipelines

Pressure pipelines laid above ground may be painted internally during the right weather, when sun on the pipeline will help to dry it sufficiently for the paint to be applied. The sprayed metal coatings mentioned above afford excellent protection, and their use is well worth serious consideration in the construction stage.

Pipelines laid on stools tend to show a first deterioration of the internal painting close to the stool, and in advanced stages the outline of the stool can be traced quite closely on the inside of the pipe by the series of fine hair cracks that have appeared in the paint work. This phenomenon is thought to be due to minute electric currents. These areas need attention more often than the rest of the pipeline, and being in the invert they are the most difficult to treat effectively.

A variety of modem paints is suitable for internal use on pipelines, but red lead followed by three coats of fairly thin bituminous paint, well worked with the brush, appears to give as good results as any, and is certainly better brushed on than sprayed.

Steel pipelines are usually steeply inclined and are extremely slippery when first de-watered due to organic growth, so that the maintenance men require ample staging and life-lines before they can embark on any inspection or maintenance work.

Pipelines require more attention externally than internally. Expansion joints, air valves, and sliding pedestal bearings should be inspected regularly, and the latter should be kept clear of grit and paint during any repainting or touching up of the pipeline.

Small rusty areas should be thoroughly cleaned, and periodically the whole pipeline, should be re painted. If a bituminous paint was used in the first case, a large number of coats can usually be applied before there is any necessity for complete cleaning down and repainting.

In the case of pipelines on steep hillsides, the hillside should receive inspection at the same time as the pipe to ensure that no large boulders are being undermined and are becoming a potential danger, in which case they may be broken into harmless sizes or underpinned with concrete pads.

For amenity reasons pipelines are sometimes painted green or brown, to blend with their surroundings, but such attempts at camouflage are seldom wholly successful. Small trees may be encouraged to grow adjacent to the pipeline but should not be allowed to grow too large or too near.

5.10 Tailraces

The turbine tailrace is usually lined with concrete for a short distance, to a point where the greatest turbulence has subsided. Up to this point maintenance is usually very light, but thereafter, in the original river bed, some minor trouble may be expected.

The tailrace lining usually consists of a base slab and concrete retaining walls. The tops of the retaining walls should be sloped to avoid any standing puddles which may result in crazing. Adequate weep holes are necessary and should be extended a few inches from the face of the wall to avoid unsightly staining.

Screens may be provided in the tailrace to prevent fish or any other animals swimming into the machines, in which case efficient tackle is necessary for lifting the screens for periodic cleaning. Though the tailrace screens are covered by the intake screens, it is surprising how much trash reaches them. It is every bit as important to keep tailrace screens clean as it is to maintain free passage through the intake screens.

Owing to fluctuating load on the turbines, conditions downstream are not as they were in nature. The greater floods may have been abolished or much reduced by the reservoir, but the equivalent of a minor flood occurs each time load is taken up quickly by the power station; so that these minor floods are frequent and have an unnaturally rapid rise and fall, which may easily lead to scouring of the river bed and banks to an objectionable degree and for surprisingly long distance downstream of the station.

It is seldom possible to forecast at the design or construction stage the extent of protective works economically necessary downstream of the station; but after a short period of operation and observation, the need for remedial works in the form of grouted or dry pitching, minor rip-rape etc., can readily be judged, and the necessary work, if small, can subsequently be constructed by the maintenance gang.

5.11 The Maintenance Squad

5.11.1 Organization

The organization of the maintenance squad varies greatly with the size and location of the project under their care and with the general policy of the Authority by whom they are employed.

The station or area superintendent is usually the prime controller of the maintenance squad. He may have in charge of the squad a maintenance engineer, to whom he can give his general orders, or he may use one of his operating assistants as part-time maintenance engineer, and he will often issue his orders direct to the maintenance foreman.

Whatever chain of command is employed, it is absolutely essential that maintenance work should be carefully planned. This planning, which should be done in conjunction with the generating, is far more difficult than is usually appreciated. There must be a plan, and it must be flexible, in order to meet real emergencies and to take full advantage of all types of weather, to suit the multitude of different types of work that has to be done. Real emergencies are rare; what are often termed emergencies arise simply from lack of planning, which has resulted in work being left undone until action has become a matter of urgency.

In addition to the plan there should be a Record Book, in which the day-to-day inspections and work of the squad are recorded in complete detail. Such a record book will in time become an invaluable guide to future planning of the squad's work, and it will show up any weaknesses there may be in the scheme as a whole, and may to better design of future projects. The record should be so kept that the cost of individual items of work can be ascertained with moderate accuracy. This cost can then be capitalized, and this figure will act as a guide to the design engineer as to how much it is worth spending on new works to avoid minor known faults.

5.11.2 Personnel

The size of the maintenance squad, like its organization, depends greatly on the size and location of the particular project and on general policy in regard to the employment of local contracting firms, if these happen to be available. Such firms can usually produce specialist labour and plant quite cheaply and readily, and their employment may well promote goodwill in the neighbourhood, a small point that is often more important than the remotely situated governing authority may realize. Any repairs, other than those of a very minor nature, on hydraulic machinery, mechanical or electrical plant, should be carried out by the maker or a specialist firm; but preferably by the maker, as he will thus have every opportunity of improving his future designs.

The basic requirements of the squad cover a very varied field. The choice of foreman is of the utmost importance; above all things he must understand men and their shortcomings, and maintain a high morale and spirit of loyalty. He must have a first-class knowledge of hydraulic equipment, and a good working knowledge of civil engineering and building practice, and he must be conscientious and observant. Local knowledge is very useful but not essential, as he will soon acquire it. The ideal man may often be found among the specialist erectors of the hydraulic machinery. Such men seem much closer to the civil- engineering aspects than are the turbine erectors, who are probably the next most suitable class.

Under the foreman there must be skilled tradesmen, preferably of the handyman type, who can turn their hands to a number of skilled jobs and at a pinch do a little manual labour when necessary. Each should also be capable of taking charge of the whole or a part of the maintenance squad.

An electrical linesman used to high voltages, as opposed to the ordinary electrician, is a very useful man in the squad; not only can he deal with wiring, switches, fuses, small motors, etc., but he can warn the others of unseen dangers. A carpenter is also essential, and he should be able to act as glazier, plasterer, and painter. The fitter should be capable of fairly advanced turning, welding, and plumbing. The labourers should be a cut above the ordinary general labourer, and amongst them there should be a good general knowledge of ropes and lifting tackles, bricklaying, concreting, dry walling, painting, and gardening, from which it will be seen that a first maintenance squad is by no means easy to recruit.

A good squad requires a team spirit. It must have pride in its work and in the many structures under its care. It must not be too large, or there will be time for idling, which quickly becomes a habit; equally, it must be large enough for there to be no question of taking in casual labour at certain times of year for the larger jobs, as this is one certain way of losing pride in good workmanship.

5.11.3 Equipment

For efficient operation it is essential that the maintenance squad be provided with ample tools of good quality. It must be remembered that the squad will be carrying out a large number of comparatively small and very varied jobs and they must have the right tools for each. On paper the tool lists will appear to be out of all proportion to the number of men, but it is a very false economy to have it otherwise.

The squad should have a central depot preferably situated at the power stations. The depot should consist of a small office, workshops, garage, tool and material store, and if the squad comprises more than 6 or 8 men there should be changing and drying room.

The fixed equipment should include a small screw-cutting lathe, drilling machine, and adequate benches for fitting, carpentry, and electrical work.

The mobile equipment should include, among the larger items, an air compressor with rock drills, breakers, grinding and drilling machines, an oxy-acetylene cutting and welding set, a small tilting-drum concrete mixer, electric and air-driven pumps, assorted lifting tackles, a supply of tubular steel scaffolding, a motor mower or mechanical scythe, powerful portable floodlights, and a light boat with outboard motor. A diving dress without the helmet will often be found useful.

Transport is a very important item to the maintenance squad, who must be completely mobile. Open trucks with detachable shelters are very useful, but in addition there must be small rugged vehicles capable of traveling over rough country road. A motor cycle is often useful for outlying inspections.

Communication is also important. The major works on a large project are usually equipped with telephones, but much of the maintenance men's work lies in the outlying parts where there is no telephone. A portable wireless transmitting and receiving set will be of considerable use on many occasions and may prove invaluable in emergencies when all telephone lines may have been swept away.

It is important that the maintenance men should have ready access to the many places on the project normally kept under lock and key. By planning the system of locks and keys in an orderly manner, it can be so arranged that the station superintendent can enter any door or locker with a single master key, while his assistants, including the members of the maintenance squad, can each open with, one key a series of appropriate locks according to their duties and trades. By this system the usual key cabinet with its multitude of different keys, some with labels and many without, becomes a thing of the past. Nor do the individuals have to remember, and carry with them, a pocketful of assorted keys when they set out on duty.

Provision should be made for the maintenance squad to hold a fairly large stock of materials, and the foreman should have authority to purchase up to a reasonable limit, so that he does not have to postpone work while awaiting delivery of bulk orders.

Regular bulk orders should be placed for such items as oils and grease in all the correct grades required by the various machines; an ample supply of paints of many types and colors are required for internal and external work; but care should be taken over the supply of perishable items such as cement, which are best obtained through a local contractor or agent as and when required. Timber, nuts, bolts, screws, and plumbing fittings will usually accumulate and develop into a very useful first-aid stock.

The squad should be encouraged to look after their transport, tools, workshops, and stores. There is a curious trait in human nature that often permits men to do excellent work and yet neglect the very tools with which they do it. The maintenance plan should provide periods for the care and maintenance of the tools.

5.12 Inspection/Maintenance Program for Penstocks

5.12.1 Inspection / Maintenance Program

A comprehensive, routine inspection/ maintenance program for the installed penstock system should be established, documented and maintained. The program must describe the inspection items included, the type of inspection/ maintenance required for each item, acceptance tolerances and criteria, the frequency of inspection/ maintenance, descriptions of possible breakdowns and other problems, and records of dates and actions taken of any inspection, maintenance, or repair. Any maintenance provisions specified by the manufacturer or fabricator must be included in the program for warranty purposes. The inspection/ maintenance instructions must include diagrams of the installed component or system.

5.12.1.1 In-service Inspection

A comprehensive well defined well documented in service inspection program is one of the means of monitoring the condition of steel pipes, liners and penstock.

Items to be inspected

The following items shall be included for inspection/ maintenance program:

- Unusual movement
- Excessive vibration
- Leakage
- Penstock age and material specification
- Penstock shell condition (inside and outside)
- Welds
- Bolts and nuts
- Expansion joints
- Air valves
- Valves or other water control systems

- Manholes and other penetration
- Anchor blocks and supports
- Coating/ linings (inside and outside)
- Instrumentation (as appropriate)
- Others, if any

Pipe Shell Inspections

The pipe shell inspection must include but not be limited to the following:

- The outer and inner surface of the pipe shells where applicable must be examined for excessive leakage, corrosion, pitting, deterioration and loading deficiencies. The inspection of leakage must be done while the pipe is watered.
- Pipe shell thickness measurements must be taken and recorded for selected location along the pipe lines or penstock line. A history of these readings gives an indication of the expected yearly decrease in the shell thickness.
- A representative portion of all structural welding performed on the inside and outside of the penstock must be examined visually for the sign of significant rusting, pitting or other structural defects.

Type of Inspection

The following inspection can be applied for inspection of pipes:

- Visual: Visual inspection is particularly effective for detecting surface defects, visual inspection must include but not be limited to the following.
  a) Inspect for rust blisters, which may indicate pinhole leaks from pitting.
  b) Inspection for liner indications, which may show several cracks in the penstock shell.
  c) Inspect the ground above a buried pipe to ensure that no trees or bushes are established; their roots may disturb the pipe and backfill. Also green growth, such as bushes, may indicate leaks in the pipe shell
- Ultrasonic Inspection
- Plate thickness can be measured accurately using drill boring or cut-out section of the pipe or penstock. The repair of the drill holes and cut-out section should be done by proper material and reliable method.

Inspection Frequency

Inspection frequency depends upon the following factors:

- Accessibility for inspection
- Consequences of a pipe or penstock failure
- Frequency of penstock or pipe watering up and dewatering.
- Climatological and environmental conditions
- PH of Water
- Amount of sediment in the water
- Pipe age
- Past history of penstock or pipe failure, if any
- Method of pipe or penstock manufacturing.

Maintenance Record

Maintenance records must be kept and should include the following items.

- Place of inspection or maintenance
- Location
- List of items repaired
- List of items replaced
- Check list of items observed
- Environmental conditions
- Recommendations for additional repairs & improvements
- Name of the person completing the inspection /maintenance.

5.12.1.2 Maintenance Operation

Each equipment and works must be checked for proper operation and for operational damage at least twice a year or as per the requirement. A maintenance procedure must be established that includes the following:

- Inspection of structures for cleanliness and ease of access
- Inspection of working parts and components
- Repair or replacement of defective or non working parts, including bolts, nuts, rubber parts (seal, gaskets)
- Flushing of equipment after maintenance to ensure a proper seal and operation.
- Evaluation of protective coatings
- Checking the operation of isolated valves and also checking that valves are left closed after completion of maintenance.
- Protecting sensitive equipment from adverse conditions.
- Scheduling maintenance during normal outages if possible

Evaluation of Penstock Condition

- Key items involved in evaluating existing penstocks include:
- Obtaining pertinent engineering documents including, but not limited to drawings, material property or classification, age, type of fabrication, fabrication and /or construction records, and operating records (including any operational test data)
- Inspecting the penstock to identify any defective areas and to determine if such defective areas are localized or global
- Determining if the defect is self-limiting or may tend to propagate further
- Establishing the mechanical properties of the penstock material including, but not limited to determining the stress versus strain relationship, yield strength, ultimate strength, chemical composition, relative ductility and fracture toughness. This information can be obtained by reviewing the original records if available, by performing destructive testing, and by performing a comparison with penstocks of similar material, construction, and operating records.
- It may be impossible to obtain or accurately establish the design allowable stress level criteria, especially for older penstocks. However, such criteria must be conservatively estimated based on the available information.
- Assessing the severity of the problem and any impact to the penstock's structural integrity and remaining service life.
- Evaluating non-technical requirements or concerns that may pose additional constraints. Some common requirements include special operational and/or maintenance requirements, accessibility, proximity to other structures or features (especially underground structures, cables, or pipes), and annual hydro plant outage times and

durations. This information should be provided by the facilities hydro-operations department.

- Developing alternative schemes at a conceptual level, including an examination of the pros and cons associated with each.
- Establishing a time frame for implementing any repair or replacement schemes.

Spare parts Data

Each material, items, components etc. must furnish spare parts data to the owner for different item of material and equipment specified, prior to final acceptance by the owner. The data must include a complete list of parts, special tools, supplies (including self life if applicable), and installation procedures, with current unit prices and supply sources. The spare parts data should be included in the inspection/maintenance procedure for each applicable component or system.

5.12.1.3 Personnel Training

The penstock manufacturer must train plant operators and maintenance staff to familiarize them with the maintenance provisions and action requirements of each pertinent system, component, and piece of equipment, if such provisions are clearly stated in the contract documents. Such training should include the specifics of what to look for, and why it is important, as well as possible breakdowns, causes, and corrective action. The training should be conducted after the penstock system is functionally completed but before final acceptance.

5.13 Periodic Inspection Schedule Format for Water Diversion and Conveyance

A sample format for a periodic maintenance schedule to maintain diversion and conveyance of designed flow in a hydropower plant has been presented in Table -5.1 below:

Table-5.1: Periodic Maintenance Schedule Format

S. No.	**Structural Elements to be maintained**	**Maintenance Schedule**		
		Year the round	**Dry Season**	**Flood Season**
Weir and Intake				
1.	Check for boulder damage	Monthly		Daily
2.	Check for floating debris at trash rack	Monthly		Daily
3.	Check for leaks, undercutting		Daily	
Regulating Sluice				
1.	Check operation	Monthly		
2.	Grease screw	Monthly		
3.	Adjust		As needed	As needed
Settling Basin				
1.	Grease flushing sluice screw	Monthly		
2.	Drain and clean	Monthly		Daily
Channel (headrace) Conduit				
1.	Inspect for leaks, overflowing	Weekly		Daily
2.	Drain and clean	Every 3 months		
3.	Grease valves	Monthly		Daily
Forebay				
1.	Clean screen	Daily		Twice daily
2.	Check screen	Weekly		
3.	Grease valves	Monthly		
4.	Drain and clean	Weekly	Monthly	Twice daily
Penstock				
1.	Visual check for flange leaks	Monthly		
2.	Repair	Every 2 years		
3.	Visual corrosion check		Once	
4.	Inspection of supports	Every six months		

CHAPTER – VI

CONCLUSIONS AND RECOMMENDATIONS

6.0 CONCLUSIONS AND RECOMMENDATIONS

These guidelines are prepared based on review, case studies and analysis of relevant available data / materials collected from field surveys of sixteen existing and under construction hydropower projects of Nepal, literature surveys of guidelines, manuals and handbooks appropriate for present study, documents from international conferences, project and design reports of dam and hydropower projects of Nepal as well as of some other countries. The guidelines, thus, have been able to incorporate almost all the structural components generally involved in design of water conveyance system of hydropower projects. During preparation of these guidelines, besides use of these materials and analytical works, judgment of the professionals engaged in the present study has also been used.

These design guidelines contain formulae and coefficients more generally used for design of water conveyance system. For cases requiring more detailed and elaborate analyses, handbooks, manuals and standards cited in places of these guidelines will have to be referred.

The guideline being a first endeavor in its kind in the context of Nepal, it’s updating and refinement in due course of time is desirable. Any relevant comments/ suggestions in this connection will be highly appreciated.

REFERENCES and STANDARDS USED

The following standards and References are relevant and useful and some of them are referred for preparing this guideline for design of penstock:

IS 2825: 1969	***Code for unfixed pressure Vessels Latest Version***
IS 11625: 1986	***Criteria for hydraulic design of penstocks, Latest Version***
IS 11639: 1986	***Criteria for structural design of Penstock (part I, II, III), Latest Version***
Welded Steel Penstock:	***Engineering Monograph No. 3 United States Department of the Interior Bureau of Reclamation***
Steel Penstocks:	***ASCE Manuals and Reports on Engineering Practice No. 79***
High Head power Plants	***Water Power Development Vol-II, By Emil Mosonye***
AWWA Manual M11	***Steel Pipe- A Guide for Design and Installation***
Volume 2 waterways	***Civil Engineering Guidelines for Planning and designing Hydroelectric Developments, published by ASCE,***
Brown J. Guthrie, editor	***Hydroelectric Engineering Practice, Volume I, Civil Engineering, Second Edition, CBS Publishers & Distributors***
Hydropower Engineering	
Handbook	***by John S. Gulliver & Roger E.A. Arndt, Published by McGraw-HILL, INC, ISBN 0-07-025193-2***
The guide to Hydropower	
Mechanical Design	***Prepared by the ASME, Hydropower Technical Committee, Published by HCI publication, ISBN 0-9651765-0-9.***