Preface

1. INTRODUCTION

The set of documents entitled *Design Guidelines for Headworks of Hydropower Projects* is one of a series of documents published by the Department of Electricity Development, Ministry of Water Resources, Government of Nepal, to establish procedural guidelines for various facets of hydropower development in Nepal. This set covers the design of headworks of run-of-river hydropower projects in Nepal.

2. DEVELOPMENT PHILOSOPHY

The guidelines presented herein have been developed along standard engineering practices incorporating considerations and requirements arising from physical and environmental conditions typical to Nepal. This development philosophy has been adopted to ensure that headworks development in Nepal is consistent with international standards and proceeds with due recognition of and emphasis on the Nepali context.

3. PURPOSE

The guidelines included in these documents are aimed at providing procedural guidance to agencies responsible for developing, operating and maintaining headworks of run-of-river hydropower projects in Nepal. They are also intended to serve as tools for the Department of Electricity Development to monitor and evaluate all engineering activities undertaken by such agencies in connection with the headworks of this class of projects.

4. COVERAGE

The guidelines cover headworks of run-of-river hydropower projects only. They encompass the common types of structures deemed suitable for headworks of both simple and pondage run-of-river schemes in Nepal.

The guidelines deal with all four phases of the headworks development cycle, viz. survey and investigations, planning and design, construction and operation and maintenance. For each phase, the guidelines cover technical, economic and environmental considerations for headworks development.

5. SCOPE

The guidelines lay down the basic objectives, typical considerations, general principles and minimum requirements for the different phases of headworks development. They do not attempt to describe details of the processes involved in various activities of these phases as such details are readily available in standard textbooks. Where required, brief information on these processes is provided, and suitable references and sources of information are cited.

6. ORGANIZATION

According to the nature of activities involved, the guidelines are divided into four parts, viz.:

Part 1: Survey and Investigations

Part 2: Design

Part 3: Construction

Part 4: Operation and Maintenance

Part 1 of the guidelines deals with various surveys and investigations required in support of planning and design of headworks. Part 2 covers the planning, detailed design and reporting requirements for various headworks components such as concrete diversion structures, embankment dams, energy dissipation structures, fishway, intakes, approach canal, desander and its flushing structures and gates. It also includes the hydraulic modeling of these components. Part 3 of the guidelines focuses on the design considerations for construction of the headworks. Likewise, Part 4 discusses the design considerations for the operation and maintenance of headworks.

7. APPLICABILITY

The guidelines presented herein apply to all government, public and private sector agencies involved in the planning, design, construction, operation and maintenance of headworks of run-of-river hydropower projects in Nepal.

8. DEVIATIONS

Deviations from the guidelines included in these documents shall be allowed provided that the procedures or criteria used in lieu of or in addition to the provisions of the guidelines are justified in writing. Any suggested changes in the guidelines that may be necessary to incorporate such procedures or criteria in future revisions of the guidelines shall be provided to the Department of Electricity Development in writing at the following address:

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Principal Contributors

On behalf of the Department of Electricity Development, the *Design Guidelines for Headworks of Hydropower Projects* has been prepared by M/S Shah Consult International (P.) Ltd., 111/44 Miteri Marg, Baneshwar, Kathmandu, Nepal. The principal contributors to these guidelines are:

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- Nepal Electricity Authority
- Bhote Koshi Power Company Pvt. Ltd.
- Butwal Power Company Ltd.
- National Hydro Power Company Ltd.
- Himal Power Company Ltd.
- Hydro Lab Pvt. Ltd.

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Topographical Survey

1. PURPOSE

Part 1A of the *Design Guidelines for Headworks for Hydropower Projects* establishes procedural guidelines, specifications and quality control criteria for topographical surveys performed in support of engineering and design of headworks of run-of-river hydropower projects. The guidelines are intended to ensure adoption of uniform and standardized procedures in topographical surveys for headworks of public and private sector hydropower projects.

2. SCOPE

The guidelines cover the use of engineering surveying techniques for topographical surveys of headworks areas. Control surveys, engineering site plan surveys, reservoir surveys and river surveys required for headworks engineering and design are discussed, and standard procedures, minimum accuracy requirements, quality control criteria, documentation and reporting procedures are outlined.

The guidelines primarily focus on conventional field surveying techniques for control survey and topographic surveys. Modern survey techniques, such as photogrammetry, are not included in the guidelines as the headworks areas of typical run-of-river hydropower projects are limited and can be surveyed using conventional techniques in an economical, accurate and timely manner. Moreover, access to the software and hardware required for the modern surveying techniques is difficult in Nepal.

3. TERMINOLOGY

Terms used in these guidelines are defined below:

Accuracy Degree to which an estimated (mean) value is comparable with an expected value.

Angular misclosure Difference in the actual and theoretical sum of a series of angles.

Azimuth	The horizontal direction of a line clockwise from a reference plane, usually the meridian.
Baseline	Resultant three-dimensional vector between any two stations with respect to a given coordinate system. The primary reference line in a construction system.
Benchmark	A permanent material object, natural or artificial, on a marked point of known elevation.
Contour	An imaginary line on the ground with all points at the same elevation above or below a specified reference surface.
Contour interval	The constant elevation difference two adjacent contour lines.
Datum	A coordinate surface used as reference for positioning control points.
Elevation	Vertical distance of a point above or below an arbitrarily assumed level surface or datum.
Latitude	Angle between the direction of a plumb line at a place and the plane of the equator.
Longitude	Angle between a fixed reference meridian, called the prime or first meridian, and the meridian of a place.
Map accuracy	The accuracy with which a map represents.
Map scale	The ratio of a specified distance on a map to the corresponding distance in the mapped object.
Mean sea level	Average elevation of the sea surface determined by continuous observation of the varying levels of the sea for as long a time as possible.
Misclosure	The difference between a computed and measured value.
Monument	A physical object set in relatively stable material or in a structure and used as an indication of the position on the ground of a survey station.

4. OBJECTIVE OF SURVEY

Topographical surveys performed at the design stage of the headworks shall be geared towards preparation of the following maps and sections of the headworks area:

a. Engineering site plan maps;

b. Reservoir area map (for diurnal pondage run-of-river projects only);

c. River profiles and cross sections.

Engineering site plan maps, profiles and cross sections shall be prepared in sufficient detail to establish a sound basis for planning, engineering and design of the headworks. Likewise, reservoir area maps shall be prepared to generate adequate data for computing the capacity and limits of submergence of the reservoir created at diurnal pondage run-of-river projects.

5. SCOPE OF SURVEY

To attain the objectives of topographical surveys listed in Section 4, the following surveys shall normally be performed in the headworks area of run-of-river hydropower projects:

a. Control survey;

b. Engineering site plan survey;

c. Reservoir survey (for diurnal pondage run-of-river projects only);

d. River survey.

The scope, standard procedures and accuracy requirements of these surveys are presented in the following sections.

6. SURVEY PLANNING

Before topographical surveys are embarked upon, survey-related field and office works shall be planned to establish the scope, extent, procedures and instrument and manpower requirements for the surveys. The planning shall be accomplished through desk studies and site visits conducted jointly by the headworks design team and the survey team.

6.1 Planning Considerations

Topographical surveys for the headworks area shall be planned taking into consideration the planning and design requirements, survey requirements over the headworks' life cycle, site conditions, existing ground control as well as economy, accuracy and project schedule.

Planning and Design Requirements

The topographical survey shall be planned to fulfill the planning and design requirements of the proposed headworks development. The plan shall ensure coverage of a sufficiently large area so that all possible locations of diversion structures, spillways, embankments, intakes, settling basins and other appurtenances are included in the survey. In addition to the minimum topographical detailing specified in these guidelines, the plan shall include suitable means to acquire any specific topographical details that may be pertinent to the planning and design of the particular headworks.

Headworks Life Cycle Requirements

The survey plan shall address the survey requirements envisaged over the life cycle of the headworks to ensure that duplicate or redundant surveys are eliminated to the maximum extent possible. In particular, the plan shall include ground control requirements stemming from surveys for layout or grade control during construction and for structural deformation studies during construction and operation. The plan shall also ensure that survey routes and monuments required for the life cycle of the headworks are not affected by construction, traffic or other forms of congestion.

Site Conditions

The survey plan shall give due consideration to existing site conditions in the selection of survey procedures, survey stations and survey lines, circuits and networks. Such conditions include the headworks area terrain, ground cover, soil characteristics, drainage patterns and structures, nearby utilities and infrastructure, wetlands, historical artifacts and other like sites. Conditions of right-of-entry to the site, or parts thereof, shall also be considered.

Existing Ground Control

Plans for establishing ground control in the headworks area shall take into account existing ground control stations in and around the area. These control stations could belong to the national geodetic network set up by the Geodetic Survey Branch of the Survey Department, GoN, or to ground control established in previous surveys of the headworks area.

Accuracy, Economy and Schedule

The survey plans shall fulfill the survey objectives in an accurate, economical and timely manner. For this purpose, the survey techniques to be used to attain the desired accuracies shall be evaluated considering the limits of errors of the survey instrument, the procedures to be adopted and the error propagation.

6.2 Planning Data and Information

Topographical surveys shall be planned based on available data and information on the proposed headworks development, topography of the headworks area and existing ground control in the headworks area. The information thus collected shall be substantiated and/or upgraded through site visits.

Information on Proposed Headworks Development

Information on the proposed headworks development plans shall be obtained from reports of previous studies. Depending on the stage of development of the headworks, such studies could include master plan, inventory, reconnaissance, pre-feasibility or feasibility studies conducted by different organizations. Information obtained from these reports shall be discussed and confirmed with the headworks planning and design team.

Information on Topography

Information on the headworks area topography shall be derived from available topographic maps, including those prepared as part of previous studies on the headworks. In particular, the following maps published by the Survey Department, GoN shall be referenced:

a. 1:125,000 scale district topographic maps with a 250 m contour interval;
b. 1:50,000 scale topographic maps with a 40 m contour interval;
c. 1:25,000 scale topographic maps with a 10 / 20 m contour interval.

The 1:50,000 and 1:25,000 scale topographic maps have been developed from 1:50,000 scale aerial photography and subsequent field verification. The 1:25,000 scale maps are presently available for most of the Terai and mid-mountain regions of Nepal. The upper mountain and Himalayan regions are covered by the 1:50,000 scale maps.

Data and Information on Existing Ground Control

Data and information on the national geodetic network present in the headworks area shall be obtained from the Survey Department, GoN. For this purpose, first, second, third or fourth order trigonometrical stations on the network shall be identified from the 1:50,000 or 1:25,000 topographic maps published by the Topographic Survey Branch of the Survey Department, GoN. Information on the coordinates, locations and monumentation of the identified stations shall then be obtained from the Survey Department, GoN.

In addition, ground control established in the headworks area during previous surveys in or around the headworks area shall be identified, and details of their stations shall be obtained from available survey reports and maps. However, the reliability of such information shall be established before the control stations are used in the proposed survey.

Information from Site Visits

During site visits, the condition and usefulness of the identified ground control stations shall be established. Issues such as accessibility to proposed survey stations, visibility, lengths of lines of sight, ground cover, need for cutting and clearing of vegetation, etc. shall be studied during the visits to confirm the proposed survey plans.

7. CONTROL SURVEY

Control survey shall be conducted in the headworks area to setup the basic framework for detailed topographical surveys for the proposed headworks development. The survey shall establish the three-dimensional point positions of carefully selected fixed monuments in the headworks area.

7.1 Connection to National Geodetic Network

Control survey performed in the headworks area shall be connected to fourth order trigonometrical stations on the national geodetic network. The connection shall be achieved either by including the trigonometrical stations in the control survey or by locating nearby trigonometrical stations through ties or spur lines.

As an alternative to connection with the national geodetic network, the control survey may be connected to control stations in the headworks area established during previous surveys. However, such interconnection may be made only if the stations are known to be connected to the national geodetic network and were established with the accuracies defined in these guidelines.

7.2 Horizontal Control Survey

Horizontal control survey shall establish the planimetric positions of control stations. These stations shall form the framework for locating contours and other details.

7.2.1 Survey Accuracy

Horizontal control surveys for different stages of headworks development shall conform to the survey accuracies listed in Table 1.

Table 1: Recommended orders of horizontal control surveys

Purpose of survey	Order
General planning and feasibility study, reconnaissance reports, generation license applications	Third Order Class II
Geotechnical investigative core borings	Fourth Order
Reservoir surveys	Third Order Class II
Site plan mapping for design memoranda, detailed design plans, contract plans and specifications	Second Order Class II

(Source: USACE EM 1110-1-1005: Topographic Survey)

The horizontal control surveys of different orders listed in Table 1 shall satisfy the relative horizontal point closure accuracy standards specified in Table 2. For checking the accuracy, the horizontal point closure shall be determined by dividing the linear distance misclosure of the survey into the overall circuit length of a traverse, loop or network line/circuit. When independent directions or angles are observed, the angular misclosures may optionally be distributed before assessing positional misclosure.

Table 2: Point closure standards for horizontal control survey

Order of survey	Point closure standard (ratio)
Second Order Class II	1:20,000
Third Order Class I	1:10,000
Third Order Class II	1:5,000
Fourth Order	1:2,500 to 1:20,000

(Based on USACE EM 1110-1-1005: Topographic Survey)

7.2.2 Horizontal Datum

Coordinates of horizontal control points shall be fixed in rectangular terms in grid meters in Northings and Eastings. The coordinates shall refer to the Modified Universal Transverse Mercator coordinate system used by the Survey Department (Table 3).

Table 3: Details of Modified Universal Transverse Mercator system

Spheroid	Everest Spheroid 1830
Central meridian	84° East
Latitude of origin	0° North
Scale factor at origin	0.9999
False coordinates of origin	Zero m at equator; 500,000 m at 84° East

7.2.3 Survey Techniques

Horizontal control survey shall be performed using one or a combination of the following method:

a. Traversing;

b. Triangulation;

c. Resection.

Of these methods, traversing shall normally be used for horizontal control of headworks areas. However, triangulation may prove more convenient for hilly areas or narrow river valleys with steep slopes.

7.2.3.1 Traversing

Traverse surveys for horizontal control shall be performed by establishing a framework of survey lines connecting a series of traverse stations in the headworks area and measuring the lengths and directions of these lines. Based on these measurements, the relative horizontal positions of the traverse stations shall be computed.

Types of Traverse

Horizontal control shall be established through closed traverse originating and terminating at control stations of equal or higher order than the order of the survey being performed. Depending on the position of control stations, loop traverses originating and terminating at the same control stations or connecting traverses beginning at one and ending at another control station shall be adopted.

Error Control

To control accumulation of error in it, the traverse shall provide a check into other known points as often as practicable. In addition, astronomical observations for position and/or azimuth shall be conducted at intervals indicated in Table 4. For third-order traverses, such observations shall also be made at abrupt changes in the direction of the traverse.

Table 4: Requirements for control traverse

Requirements	Second-order traverse	Third-order traverse
Horizontal angle		
Instrument	0.2” – 1.0”	1.0”
Repetitions	6 – 8	2 – 4
Rejection limit	4” – 5”	5”
Number of courses between azimuth check		
Steel tape	25	35 – 50
EDM	12 – 16	25
Azimuth closure		
Standard error	2.0”	5.0”
At checkpoint	3” per station or (10”)$N^{0.5}$	5” per station or (15”)$N^{0.5}$
	(N = number of stations carrying azimuth)	

(Based on USACE EM 1110-1-1004: Geodetic and Control Survey)

7.2.3.2 Triangulation

Triangulation shall be performed by establishing a network of interconnected or overlapping triangles in the headworks area and precisely measuring all vertex angles of the triangles and a baseline distance on the network. The sides of the network triangles, and subsequently the horizontal coordinates of the triangulation stations, shall be calculated from these angle and baseline measurements.

Triangulation Networks

The triangulation network shall originate from and tie into existing control stations of equal or higher order than that of the survey being performed. As far as possible, it shall allow computations to be performed through two independent and well-conditioned routes. Its sides shall be of comparable lengths, and it shall not contain very long lines of sight.

Error Control

To control accumulation of errors in the computed lengths and azimuths of the sides of the network, subsidiary bases on the triangulation shall also be measured at suitable intervals. Astronomical observations for azimuth and longitude shall also be made at certain stations to control accumulation of errors.

7.2.3.3 Resection

Resection shall be used to fix horizontal control when existing control stations cannot be occupied for making observations required for doing so. Through this method, horizontal control shall be established at the position occupied by the surveyor by making observations to three other stations or points of known coordinates.

Resection Figure

The control points observed for resection shall be carefully judiciously selected so that they form strong geometric figures. For this purpose, the angles between control stations at the point of observations shall be maintained between 60º and 120º of arc.

Error Control

If lines of sight allow, more than three control stations shall be included in the resection figure to check the accuracy of resection. If possible, at least one of the control stations shall be occupied to check on the computations and to increase the positioning accuracy.

7.2.4 Survey Equipment

Angular measurements for horizontal survey shall be performed with a repeating theodolite having an optical micrometer with a minimum least count resolution of 6” or a directional theodolite having an optical micrometer with a least count resolution of one arc second. For distance measurements, an Electro-magnetic Distance Measuring (EDM) device shall be used. Optionally, a total station having angular and distance measurement capabilities equal to, or better than, those of the theodolites and EDM shall be employed.

Survey equipment used for horizontal control survey shall be tested, calibrated and adjusted. These activities shall be conducted regularly, or as required, according to manufacturer’s specifications.

7.2.5 Redundancy in Measurements

During measurements, redundant linear and angular observations shall be taken to reduce errors. At least four sets of horizontal angle readings shall be observed on each instrument face to minimize collimation errors. The readings shall also be observed at different zeros to minimize the graduation errors. With EDM distance measurements, a minimum of two readings shall be taken at each setup.

7.2.6 Field Documentation

Field data collected during horizontal control survey shall be systematically recorded and supplemented with adequate notes and sketches. For each station occupied, the instrument height, target heights and observed distances and angles shall be carefully recorded in a field book as the measurements are made in the field. A description of the occupied stations, including their monument type, general location and the type of ground, shall be included in the field notes. Where required, a sketch of the location of the station relative to existing physical features and reference ties shall also be made in the field notes.

Field control charts shall be maintained for horizontal control points. For triangulation, a field triangulation chart showing trigonometric stations, existing triangulation stations in the headworks area and new stations and their connecting rays shall be plotted. For traverse surveys, a field traverse chart showing traverse stations and lines/circuits shall be prepared. The charts shall be at plotted at one-half or one-fourth of the scale of the engineering site plan survey.

7.3 Vertical Control Survey

Vertical control survey shall establish the elevations of control stations and benchmarks in the headworks area. These stations or benchmarks shall serve as points of departure and closure for leveling operations and as a reference framework for determining elevation differences in the headworks area.

7.3.1 Survey Accuracy

Vertical control surveys performed during different phases of headworks development shall conform to the survey accuracies listed in Table 5.

Table 5: Recommended orders of vertical control surveys

Purpose of survey	Order
General planning and feasibility study, reconnaissance reports, generation license applications	Third Order
Geotechnical investigative core borings	Third or Fourth Order
Reservoir survey	Fourth Order
Site plan mapping for design memoranda, detailed design plans, contract plans and specifications	Second or Third Order

(Source: USACE EM 1110-1-1005: Topographic Survey)

The vertical control surveys of different orders listed in Table 5 shall satisfy the relative vertical point closure standards specified in Table 6, where K is the length of the survey line or circuit in km. For comparison with these standards, the accuracy of a vertical control survey shall be determined by the elevation misclosure within a level section or a level loop.

Table 6: Point closure standards for vertical control survey

Order of survey	Point closure standard (mm)
Second Order	$\pm 6\sqrt{K}$
Third Order	$\pm 12\sqrt{K}$
Fourth Order	$\pm 24\sqrt{K}$

(Source: USACE EM 1110-1-1005: Topographic Survey)

7.3.2 Vertical Datum

The vertical coordinates of control points in the headworks area shall refer to the mean sea level.

7.3.3 Survey Techniques

Vertical control shall be fixed by one or a combination of the following methods of leveling:

a. Trigonometric leveling;

b. Differential leveling.

Trigonometric leveling shall be used for establishing vertical control points if the headworks area terrain is steep and undulating and a contour interval 5 m or more is desired. If the headworks area is undulating or hilly and the contour interval is 5 m or larger, vertical control shall be provided by trigonometric leveling connected to differential leveling where possible. However, if the headworks area is relatively flat and the contour interval is 1 m or smaller, vertical control shall be established by differential leveling.

7.3.3.1 Trigonometric Leveling

Trigonometric leveling shall be performed to determine the difference in elevation between stations by observing the vertical angles and the horizontal or slope distances between them and applying the fundamentals of trigonometry. The leveling shall originate from and tie into existing control, and the elevation of any station on the level line or circuit shall be fixed by algebraically summing its elevation difference from the existing control with the elevation of that control.

Error Control

The effects of curvature and refraction on the observations of trigonometric leveling shall be prevented by avoiding long lines of sight between stations. The lines of sight shall be limited to 300 m when electronic survey equipment is used. Where long sights cannot be avoided, proper corrections to curvature and refraction effects shall be applied linearly.

Errors in trigonometric leveling due to atmospheric refraction shall also be eliminated by taking reciprocal observations between two stations when the refraction is steady. For this purpose, vertical angles shall be observed between 1300 to 1600 hours when the refraction is much less variable than in the morning or late afternoon.

7.3.3.2 Differential Leveling

Differential leveling shall be performed to establish the difference in elevation between stations by measuring their vertical distance with respect to a horizontal line of sight. All levels shall start from and close on existing control, and the elevation of any station on the level line or circuit shall be determined by algebraically summing its elevation difference from the control with the latter's elevation.

Error Control

The effect of curvature and refraction on the lines of sight shall be eliminated by avoiding long sights and by maintaining balanced fore and back sights from the instrument station. Lines of sight shall generally not be longer 100 m. Long lines of sights shall be split into smaller lengths through intermediate setups of the level instrument.

In situations where long lines of sights between two stations cannot be avoided, reciprocal readings shall be taken from the stations to eliminate the effects of curvature, refraction and collimation error. Alternatively, necessary corrections to level observations shall be made to account for curvature and refraction.

7.3.4 Survey Instrument

Trigonometric leveling shall be performed using a directional theodolite or total station. For second-order differential leveling, a precise level or total station and precise leveling rods shall be used. A semi-precise level, such as the tilting Dumpy type, three-wire reticule or equivalent, and semi-precise leveling rods shall suffice for third-order differential leveling. The selection of spirit level instrument for lower-order leveling shall be consistent with the required control point accuracy. Less precise level instruments, such as the Fennel tilting level, dumpy level, Wye level or equivalent, and a stadia rod with least readings of 1 cm may be used.

All instrument used for leveling shall be tested, calibrated and adjusted at regular intervals. These activities shall be carried out in accordance with the manufacturer's specifications.

7.3.5 Redundancy in Measurement

In trigonometric leveling, redundant linear and angular measurements shall be taken to reduce errors. At least four sets of vertical angle readings shall be observed on each face of the instrument to minimize collimation errors. Likewise, at least four reciprocal readings shall be observed for slope or horizontal distances.

7.3.6 Field Notes and Control Charts

Field data collected during vertical control survey shall be systematically recorded and supplemented with adequate notes and sketches. Complete notations or sketches shall be made to identify level lines and side shots. A short description of the course of the level line shall be entered in the field book. Entries shall be made in the book that give the references to the traverse notes and other existing data used for elevations. In addition, a complete description of each point on which an elevation is established shall be recorded in the field book adjacent to the station designation.

Field control charts shall be maintained for vertical control points. For leveling, a field leveling chart shall be maintained on an existing large scale map of the area. Higher level lines and benchmarks shall be clearly marked on this chart in a distinctive color. Likewise, all new level lines and their alignments, along with benchmark positions, shall be plotted clearly on the chart.

7.4 Monumentation

Monuments for horizontal and vertical controls shall be provided in the headworks area to preserve their planimetric positions and elevations, respectively. These monuments shall be stable to preclude the introduction of errors in subsequent surveys that are based on them. They shall also be capable of surviving the intended period of their use. These objectives shall be achieved through proper site selection, installation and documentation of the monuments.

7.4.1 Site Selection

Monuments shall be sited at appropriate locations in the headworks area. These locations shall be selected considering the security, functionality and stability of monuments.

Security

Monuments shall not be located in areas that are susceptible to damage or destruction due to construction, erosion, undercutting or collapse. River banks, flood plains and unstable areas shall be avoided. However, sites that provide natural protection or permanent and stable manmade features shall be considered for establishing monuments.

Functionality

Monument sites shall be readily accessible and capable of being conveniently occupied for observations. They shall be easily identifiable with reference to fairly permanent objects in their vicinity. They shall also be visible from as large a part of the headwork area as possible. For this purpose, areas with dense vegetation shall generally be avoided.

Stability

Monuments shall not be located in regions that are likely to be affected by geological and soil activities. Sites susceptible to slope instability, subsidence, frost heave, volume change and poor drainage shall be avoided. To safeguard them against subsidence and instability, monuments shall preferably be located on sound and intact bedrock in stable and firm land. Pockets of unstable ground, such as those around caverns and underground structures, and fractured, fissured and weathered bedrocks shall be avoided. Where bedrock is not available, monuments shall be established in coarse-grained soils to avoid the effects of frost heave, volume change and poor drainage commonly encountered with fine-grained soils.

Monuments shall not be placed near water reservoirs and large rivers where variable water levels can cause the ground to rise and fall due to rebound and compression of the soil. If possible, a distance of a few hundred meters between the monuments and the boundaries of these sources of ground activity shall be maintained.

Integral parts of stable manmade structures may also be used for positioning monuments. Large concrete, steel or masonry structures resting directly on bedrock, deep piles or piers shall be preferred for this purpose. Structures on other types of foundations may be chosen for monuments only if the structure's age exceeds five years. However, small structures like culverts, platforms, retaining walls, etc. shall not be used for monumentation.

7.4.2 Types of Monuments

The type of monument at a particular station shall be selected based on local site conditions. Monuments installed on rock or manmade structures and in soil shall meet the requirements discussed below.

Monuments on Rock and Manmade Structures

Monuments on rock or manmade structures shall be marked by drilling a 25 mm diameter, 100 mm deep hole in the rock or structure and grouting a brass nameplate in it such that its top is flush with the surrounding. Alternatively, the monument shall be marked with a 5 mm diameter, 20 mm deep hole drilled into the rock with a circle engraved around it. For more precise work with EDM instruments, the drilled hole shall have a diameter 1 to 2 mm and a depth of 3 to 4 cm, and a copper or brass wire shall be plugged into it and made flush with the surrounding surface.

Monuments in Soil

Monuments in soil shall be marked by pouring fresh concrete in a 500 mm deep, 200 mm square excavation in the ground and fixing brass nameplates on its top. Alternatively, a 1 m long, 25 mm diameter pipe shall be driven vertically into the ground and its center shall be taken as the actual mark. As another option, a large stone, with a circle and dot cut on it, shall be embedded about 1 m into the ground, and a similarly marked stone with its dot aligned vertically with the dot on the lower stone shall be placed flush with the top of a 500 mm high, 3 m square surrounding platform.

7.4.3 Monument Names

Monuments shall be named based on a systematic scheme. A horizontal control monument shall be identified by an intelligible name stamped on its nameplate. The assigned name shall be concise and in itself be descriptive and/or indicative of the general location of the control point. Likewise, a vertical control point shall be identified by a number or an alphanumeric symbol stamped on the respective nameplate or inscribed on the monument.

7.4.4 Monument Documentation

Each permanent monument established in the headworks area shall be fully documented to record its position, description and related data. Descriptive data and other information available in the field shall be recorded at the station site, and all other applicable data shall be added as this information becomes available.

Based on the information, a description card containing the following information shall be prepared for each permanent monument:

a. Name or station designation of the monument.
b. Name of the community, town or city near the monument.
c. Name of village development committee or municipality, district and zone.
d. Month, day, and year the monument establishment.
e. Elevation of the top of the monument.
f. Exact latitude and longitude of the monument.
g. Horizontal and vertical datum referenced.
h. Order and class of accuracy of horizontal and vertical stations.
i. Type of monument and its details.
j. Description of the monument including, but not limited to, the following:
    i. A short narrative providing specific directions on reaching the monument from a readily locatable landmark.
    ii. Exact location of the monument at the station site, with distance and direction from at least three reference objects in the immediate vicinity.
    iii. Vertical reference of the monument in terms of the monument's distance above, below or level with a nearby reference object or ground surface.
    iv. Information stamped on brass nameplates cast at the station.
k. A sketch of the monument location relative to existing physical features and reference ties.
l. Photograph of the station with surrounding area.

7.5 Survey Computations

Two independent sets of computations shall be performed on the control survey data, and the sets shall be compared to check any gross disagreement. Both sets of computations shall be signed and dated by the persons responsible, and the completed computations should be scrutinized by a responsible officer.

7.6 Documentation

On completion of survey computations, the following records of the control survey shall be prepared:

a. History sheet of control surveys.
b. Description of stations/benchmarks.
c. List of coordinates/elevations.
d. Traces of triangulation/traverse/leveling charts.

The history sheet shall include a short narrative account of the control survey, mentioning the object of the control survey, the instruments and signals used and other facts of interest.

All basic data collected during the control surveys, and all computations made as a part thereof, shall be preserved and filed systematically so that they are readily available and easily understood. The records shall be classified job-wise and kept safe. All original records and computations shall be preserved carefully.

8. ENGINEERING SITE PLAN SURVEY

Engineering site plan survey of the headworks area shall be performed to determine the planimetric location and topographic relief of features in three dimensions. This survey shall result in preparation of detailed large-scale site maps for conceiving, justifying, designing and constructing the headworks.

8.1 Map Scale

The scales for engineering site plans prepared during different stages of development of the headworks shall be selected from the range of scales recommended in Table 7. For a given stage of development, the smallest map scale that can provide the necessary details in an economical manner shall normally be chosen from the recommended range. However, a larger scale may be adopted if any other larger-scale map uses of the mapped area are anticipated for the headworks development.

Table 7: Recommended scales for engineering site plans

Purpose of survey	Target map scale
Planning, feasibility study, generation license applications	1:1,000 – 1:5,000
Reservoir survey	1:5,000 – 1:10,000
Site plan mapping for design memoranda, detailed design plans, contract plans and specifications	1:250 – 1:500

(Source: USACE EM 1110-1-1005: Topographic Survey)

8.2 Contour Interval

Contour intervals for engineering site plans shall be selected from the ranges recommended in Table 8.

Table 8: Recommended contour intervals for engineering site plans

Purpose of survey	Contour interval (m)
Planning, feasibility study, generation license applications	0.50 – 2.00
Geotechnical investigative core borings	0.25 – 1.50
Reservoir survey	1.00 – 5.00
Site plan mapping for design memoranda, detailed design plans, contract plans and specifications	0.25 – 0.50

(Based on USACE EM 1110-1-1005: Topographic Survey)

For a particular stage of headworks development, the contour interval shall be judiciously selected considering the following factors:

a. Desired accuracy of the depicted vertical information.
b. Relief of the headworks area.
c. Cost of field work and fair mapping.
d. Other practical uses for the intended map.

If a specific vertical tolerance is to be satisfied, the contour interval shall be selected as a direct proportion to the tolerance (refer Table 10). Larger contour intervals shall be adopted for areas with steep slopes in order to make the map more legible. In flatter areas, smaller intervals that do not interfere with planimetric details located on the map shall be chosen. Contour intervals shall be chosen to achieve economical field works and mapping without compromising on the map accuracy. For ease of mapping and legibility, intervals resulting in contours closer than 2 mm shall generally be avoided. Although the largest possible contour interval is desirable, smaller intervals may be selected if any other map uses of the mapped area are anticipated for the headworks development.

8.3 Mapping Standard

Engineering site plan maps prepared for different stages of headworks development shall conform to the horizontal and vertical mapping accuracy requirements listed in Table 9 and Table 10, respectively. In these tables, the horizontal map accuracy standards are expressed in terms of two-dimensional root mean square radial positional errors while the vertical map accuracy standards are expressed in terms of one-dimensional root mean square elevation errors. The map location and elevation tolerances in these tables are defined relative to two adjacent points within the confines of a map sheet, not to the overall headworks area.

Table 9: Recommended horizontal mapping accuracy standards

Purpose of survey	Map scale	Feature location tolerance (m)
General planning and feasibility study, generation license applications	1:1,000	0.50
	1:5,000	3.00
Geotechnical investigative core borings	1:5,000	1.50 – 3.00
Reservoir survey	1:5,000 to 10,000	2.00-5.00
Site plan mapping for design memoranda, detailed design plans, contract plans and specifications	1:250	0.015
	1:500	0.30

(Source: USACE EM 1110-1-1005: Topographic Survey)

Table 10: Recommended vertical mapping accuracy standards

Purpose of survey	Contour interval (m)	Feature elevation tolerance (m)
General planning and feasibility study, license applications	0.50	0.15
	3.00	0.60
Reservoir survey	5.00	1.25
Site plan mapping for design memoranda, detailed design plans, contract plans and specifications	0.25	0.003
	0.50	0.15

(Source: USACE EM 1110-1-1005: Topographic Survey)

For computing the accuracy of a map, at least three pairs of well-defined points within the map boundary shall be selected so that their combinations result in at least 15 error values. Points selected for this purpose shall be capable of functionally maintaining a given accuracy tolerance between themselves, e.g. adjacent property corners, adjoining buildings, bridge piers or approaches or abutments, etc. The coordinates of these points shall be obtained from scaling the finished map and from field measurements performed using survey methods superior to the methods used for constructing the map. The horizontal error shall then be obtained by taking the root mean square of the squares of the errors in latitude and departure. Likewise, the vertical error shall be computed from the root mean square of the squares of the errors in elevation.

8.4 Survey Techniques

Engineering site plan survey shall be carried out using one of the following methods:

a. Plane tabling.

b. Tacheometry.

Of these methods, plane table methods may be used for mapping headworks areas with limited lateral extent. However, tacheometric survey procedures for mapping shall generally be preferred in view of their speed and accuracy.

8.4.1 Plane Tabling

Plane tabling shall be performed to map the headworks area using graphical methods. For this purpose, the headworks area shall be divided into several sections so that each section may be conveniently accommodated and surveyed on the plane table. The sheet limits for a particular section shall be indicated by rectangular projection lines, and existing horizontal and vertical control stations shall be plotted on the plane table. Plane tabling shall start from these control stations, and the survey for details and contours shall be conducted together in the field.

8.4.1.1 Detailing

Within a particular plane table section, details shall be located graphically using intersection and radiation techniques. Details shall be located for the following features:

a. Works of man, such as buildings, roads, bridges, dams and canals.
b. Natural features, such as streams, lakes, edges of wooded areas and isolated trees.
c. Relief.

On large-scale maps, the true shape of features shall be represented to scale. However, on small-scale maps, buildings and other features shall be portrayed by means of standardized topographic symbols centered on the true position but drawn larger than the scale of the map.

8.4.1.2 Contouring

For large contour intervals, contours may be surveyed by plane tabling techniques; however, for small contour intervals, contours shall be surveyed by interpolation based on a network of spirit-leveled spot heights.

Detail points and elevations for contouring shall usually be located at key points of distinct changes in ground slope or in the direction of a contour. Such key points shall be located at the following positions:

a. Hill or mountain tops.
b. On ridge lines.
c. Along the top and foot of steep slopes.
d. In valleys and along streams.
e. In saddles between hills.

Contour lines shall be drawn on the map by logical contouring. Ground elevations shall be determined at key points of the terrain, and these positions shall be plotted on the plane table sheet. Contour lines shall be sketched after a number of key points have been located and plotted. Every fifth contour line shall be drawn heavier than the other contour lines, and the elevations of these heavier lines shall be shown at frequent intervals.

8.4.1.3 Records and Computations

Plane table stations shall be designated in a systematic manner. Such designations shall be written on the plane table sheet and also entered in a suitable notebook. A brief description of objects sighted may be noted on the plane table sheet and written along the line of sight, and their complete description shall be recorded in the notebook. The notebook shall also contain the vertical angle records.

8.4.1.4 Map Preparation

Maps prepared through plane tabling shall be finished with standard symbols and inked in proper colors. The plane table sections shall be kept free from color washes which tend to confuse the details. Separate color trace and height trace shall be prepared on tracing cloth covering the area of each plane table section to facilitate fair mapping later on.

8.4.1.5 Plane Table Equipment

A plane table, telescopic alidade and stadia rod shall be used to locate details and contours in undulating areas. However, a spirit level shall be used for spot leveling in hilly areas.

8.4.1.6 Error Control

While defining plane table sections, care shall be taken to ensure that a sufficient number of horizontal and vertical control points are present all along the borders of each plane table section so that the desired accuracy of survey is attained at its edges also. At any station, the plane table shall be oriented with respect to a horizontal control station, and the orientation shall be verified by sights to additional visible horizontal control stations. To prevent errors due to movements, the orientation of the plane table shall be checked from time to time while making observations and on completion of the work at each station.

In general, the principal stations from which the greatest number of vertical angles are to be taken shall be connected by means of reciprocal vertical angles taken under differing conditions. In measuring important vertical angles, such as those to other stations or to points on level lines, all readings shall be checked by reversing both the level and telescope and by using different positions of the vertical arc. Proper corrections for refraction shall be applied whenever appropriate.

8.4.1.7 Check for Accuracy

Before the survey team withdraws from the headworks area, the plane table survey shall be tested on the ground by field inspection. Discrepancies found in this test shall be promptly rectified.

8.4.2 Tacheometry

Tacheometric procedures shall be used map the headworks area through measurement of horizontal and vertical distances of points by optical means. The distances shall be measured through radial surveys from setup points established through traversing.

8.4.2.1 Tacheometric Traversing

The tacheometric traverse for establishing setup points for subsequent radial surveys shall be run with reference to existing control in the headworks area. The traverse stations, which will form the setup points for radial survey, shall be chosen at commanding positions in the headworks area. The elevations of these stations shall be established using conventional leveling techniques instead of the trigonometric values determined from tacheometry.

8.4.2.2 Radial Survey

Radial surveys shall be performed from control stations or traverse stations. For obtaining mapping information, several radial lines making different angles with either the magnetic meridian or with the first radial line shall be set out. On each radial line, readings shall be taken on leveling staff kept at different ground points chosen such that the approximate difference in elevation between two consecutive points is less than the contour interval.

8.4.2.3 Data Collection

Systematic field procedures shall be adopted to collect field data. These procedures shall be designed to provide uniformity in data collection and to avoid confusion in map generation.

Code for Field Data

Whether data are recorded manually or electronically, a coding scheme for recording field data shall be developed before the field surveys start. This scheme shall identify points for locating the various topographic features in the headworks area to ensure correct plotting and contour interpolation. If maps are to be generated using computer programs, the coding scheme shall be designed to enable the selected program to interpret the recorded data without ambiguity and create a virtually finished product.

Data Collection

Data collection shall be performed in a sequence that eases the processing and plotting of field data. If computer programs are used for these activities, the sequence shall be defined in conformity with the specific requirements of the program. In general, however, the following sequence for data collection shall be followed:

- Data on planimetric features, such as roads, buildings, etc.
- Additional data needed to define the topography.
- Data on break lines, such as road edges and streams, which segregate regions of contour interpolation.
- Additional definition of ridges, vertical, fault lines and other features.

To facilitate map preparation using computer programs, data points for linear features, such as electricity lines, road edges, etc., shall be obtained consecutively without shooting at other topographic feature in between. Data for features with more than one linear edge, e.g. roads, shall be collected by surveying one edge at a time.

The extent of data collected in any part of the headworks area shall be consistent with the nature of the terrain and the accuracy requirements. A few extra shots shall be taken at locations where generation of a good contour map requires greater details and elevations.

Sketches and Field Notes

During the field survey, planimetric features shall be sketched, photographed or filmed to ease proper deciphering of field data. Sketches shall be adequate to allow office personnel involved in map generation to confirm the correctness of feature codes. Where required, the sketches shall be detailed enough to communicate complex information directly to the design engineer without lengthy discussions. Miscellaneous descriptive notes may also be shown on the sketches for later addition to design files.

To establish complete records, survey conditions, unplanned procedures and other pertinent information shall be recorded in the field notes. These records shall be maintained even when automatic data loggers are used for data collection.

8.4.2.4 Error Control

To ensure that any outside influences do not degrade the instrument setup, observations to control stations shall be measured and noted immediately after the instrument is set up and leveled. In making observations for an extended period of time at a particular instrument location, the control points shall be observed from time to time to ensure that any data observed between the control shots are good. As a minimum, both vertical and horizontal controls points shall be observed at the beginning of each instrument setup and again before the instrument is picked up.

In order to minimize pointing errors, more than one set of angle and distance measurements shall be taken, and their average value shall be used. This process shall especially be adopted during adverse weather conditions and while sighting points at steep angles or points at distances in excess of 200 m.

Errors due to uneven heating of instrument by direct sunlight shall be prevented by setting the equipment in shaded spots. Instrument locations that vibrate and could cause a total station’s compensator to be unstable shall be avoided. When sighting points a single time for elevations, the instrument shall be checked regularly for collimation errors.

8.4.2.5 Data Reduction

Topographic survey data shall be reduced in the field. The reductions shall preferably be carried out using programmable calculators or computers.

Before data is reduced, the field input of data, especially the instrument locations, azimuths to back sights, elevation of benchmarks and the rod heights, shall be checked against field notes. The data shall be scanned for any information that seems to be out of order. Any information flagged in the field as being in error shall be eliminated; however, such editing shall be properly documented.

Following data checking and editing, the control data shall be processed. A short report of the data collected in the field shall be produced. Benchmark elevations shall be checked to ascertain that the given elevations are the calculated elevations and that the coordinates of the back sights and foresights are correct. Any errors in the data detected during this process shall be corrected immediately at the site.

8.4.2.6 Map Preparation

After field data have been reduced and compiled, engineering site plans shall be generated electronically using computer-aided design and drafting software packages. In this process, planimetric features and topographic elevations shall be separated onto different layers and depicted at the desired scales. These spatial data layers shall contain descriptor information identifying the original source target scale and designed accuracy of the maps.

The person responsible for the field work shall be involved during the initial phases of map generation. To ensure that the final map matches the actual conditions, this person shall also review the completed map.

8.4.2.7 Survey Equipment

Tacheometric surveying shall be conducted using a plain or electronic theodolite fitted with stadia diaphragm and capable of providing staff readings against all three hairs. However, an electronic total station fitted with a data collector shall be preferred for this purpose.

All survey equipment used for tacheometric surveys shall be tested, calibrated and adjusted regularly, or as required, in accordance with manufacturer's specifications. For total stations, the following tests shall be performed:

a. Adjustment of the electronic tilt sensor and the reticle of the telescope.
b. Calibration of atmospheric temperature and pressure measurement instruments.
c. Alignment of optical plummet or tribrachs.
d. Adjustment of the leveling bubble of prism pole.

8.4.2.8 Training of Field Survey Crew

The field crew responsible for survey shall be well-versed with the automated processes for map generation so that they can gather field data appropriately. For this purpose, the crew shall be cross-trained in office procedures for data reduction, contouring and detailing.

8.5 Map Compilation and Drafting Specifications

The engineering site plan shall be compiled at the required scale on A1, A2 or A3 size paper. The layout and contents of the final maps shall comply with the specifications listed below.

Layout

The headworks area maps shall be arranged to meet the following minimum requirements:

a. The coordinate grid system for the maps shall be established on the Modified Universal Transverse Mercator system (refer Table 3). Grid ticks shall be placed on the map sheets with coordinate values properly annotated and shown at the top and right edge of each map sheet.

b. Multiple map sheets shall contain an index of the sheet layout oriented north relative to their true position. Match lines/match grids shall be provided and properly labeled such that each sheet may be joined accurately to adjacent sheets.

c. Symbology used on the map sheets shall be clearly indicated on each sheet. Standard symbols, preferably those used in topographic maps published by the Department of Survey, GoN, shall be used.

d. The title block, sheet index and legend shall be placed on the map sheets to an appropriate size and arrangement. The title block shall include the name of the project development agency, project name, date and scale. It shall also contain the name and logo of the agency performing the survey.

e. An accuracy statement shall be published in the notes of each map sheet stating that the finished map meets the horizontal and vertical map accuracy standards of these guidelines.

Control Features

All horizontal and vertical controls shall be plotted on the map to an accuracy of relative to their true position. Primary control set to control construction phases shall be labeled as such.

Topographic Features/Contour Development

Representation of topographic features and contours in the maps shall meet the following requirements:

a. Contours shall be legible and drawn sharp and clear as solid lines. Every fifth contour (index contour) shall be accentuated as a heavier line than the intermediate four. Half interval supplemental contours shall be added as required. Labeling or numbering of contours shall be placed so the elevations are readily discernible. Labeling of intermediate contours may be required in areas of low relief.

b. Turning points that define drainage channels, ditches, etc. shall be consistent in depicting correct alignment and direction of drainage.

c. Spot elevations shall be shown on the maps at the following locations:
   - Water surfaces on shorelines of lakes, reservoirs, ponds and the like.
   - High and low points at hilltops and depressions.
   - Intersections and along center lines of roads and where applicable.
   - Tops and bottoms of vertical walls and other structures.
   - Center line of end of bridges.

   Ground spot elevations shall sufficiently supplement contoured elevations. Spot elevations shown on the map sheets shall be accurate to the designated contour interval.

d. Digital elevation models shall be generated by grid or trace controlling methods on a network of random points supplemented with break-line points to properly establish the terrain model.

Planimetric Feature Data Detailing

The maps shall contain all planimetric features encountered within the headworks area and compatible with engineering site plans. These shall include, but not limited to, the following:

- Buildings and pertinent structures, including names of major buildings and landmarks.
- Streets, roads and highways, including their names, pavement width, type and surface condition.
- Sewer service lines, including major systems and appurtenances.

- Utility systems, surface and subsurface, including all appurtenances like communication, water, fuel, electric, telephone, overhead power lines, transmission pipelines.
- Storm drainage features and structures, bridges, culverts, piers, spillways and channel systems.
- Timbered areas, landscapes and individual trees that are recognized as such.
- Recreation areas.
- Cremation grounds, cemeteries.

9. RESERVOIR SURVEY

Reservoir survey shall be carried out in headworks areas of diurnal pondage run-of-river schemes. This survey shall be aimed at estimating the capacity of the reservoir along different contour levels and determining the extent of area that will be submerged consequently. The reservoir area map generated from this survey shall also serve as a base for subsequent sedimentation studies performed during the operation of the headworks.

9.1 Survey Control

Reservoir surveys shall be performed with respect to existing horizontal and vertical control established in the headworks area. Two permanent benchmarks connected to the nearest existing benchmark shall be established to provide height datum for all leveling and traverse heighting of the reservoir survey.

9.2 Survey Techniques

Plane tabling or tacheometric methods shall be employed for reservoir surveys. Procedures for conducting these surveys shall comply with the requirements of Sections 8.4.1 and 8.4.2, respectively.

9.3 Detailing and Contouring

Reservoir survey shall extend up to +10 m above the maximum water level of the proposed pondage. It shall cover all topographical features and cultural details in the reservoir area. Towns and village sites, cultivated and forest lands, power and overhead communication lines, old and new mines and quarry sites, etc. shall be surveyed and mapped.

Contours in the reservoir area shall be determined with sufficient accuracy to allow accurate calculation of the reservoir capacity. However, the highest contour near the top of the dam shall be surveyed very accurately through differential leveling.

9.4 Full Reservoir Level Marking

As part of the reservoir survey, the limit of the full reservoir level shall be accurately marked on the ground at suitable intervals. In areas where leveling is possible, this limit shall be marked by surveying the full reservoir level contour with a clinopole. When ground conditions do not permit this, a height traverse shall be run from a benchmark fixed by leveling and adjusted before the final contour position is arrived at.

10. RIVER SURVEYS

River surveys shall consist of longitudinal profiling and cross-sectional survey of the river upstream and downstream of the proposed headworks site.

10.1 Longitudinal Profiling

Longitudinal profile of the river in the headworks area shall be prepared by leveling at intervals of 20 m or less along the fair weather deep channel. On the upstream side, the profiling shall extend from the axis of the proposed diversion structure up to the minimum of the following:

- Anticipated point of back water effect.
- Maximum back water level + 10 m.
- Any headworks situated upstream of the proposed headworks.

On the downstream side, the profiling shall extend from the axis of the proposed diversion structure up to the minimum of the following:

- Five km from the axis of the proposed diversion structure.
- Up to the nearest downstream headworks.

10.2 Cross-sectional Survey

River cross-sections shall be surveyed along the axis of the diversion structure and upstream and downstream of it. The cross-sectional surveys shall be performed by leveling at intervals of 20 m or less.

On the upstream side, river cross-sections shall be surveyed at intervals of 100 m for a distance of 2 km from the axis of the diversion structure and thereafter at intervals of 1 km up to the initial point of the longitudinal profile. Each cross-section shall extend on either side of the firm bank of the river up to the minimum of the following:

- Maximum water level + 10 m;
- One kilometer.

On the downstream side, the cross-sections shall be surveyed at intervals of 100 m for a distance of 500 m from the axis of the diversion structure and at intervals of 1 km up to the end of the longitudinal profile, depending upon the change in profiles and plan form of the river. Each cross-section shall extend up to the historical highest flood level + 5 m on either side of firm bank of the river.

10.3 Preparation of Profiles and Cross-sections

The following items shall be indicated in the longitudinal profile:

a. Date of survey of the particular reach and water level on that day.
b. Deep pools and rapids, rock outcrops, etc.
c. Maximum historical observed highest flood level.

The following items shall be indicated in the cross-section:

a. Date of survey and the water level on that day.
b. Minimum water level.
c. Maximum historical/observed water level.
d. Rapids, rock outcrops, etc.

The longitudinal profiles and cross-sections shall be plotted at a horizontal scale of 1:2,500 and a vertical scale of 1:100.

11. SURVEY REPORT

Upon completion of the survey, a survey report containing the following shall be prepared:

a. Methodology of survey
b. Instruments used.
c. Description of national control points taken for reference.
d. Description of local control points.
e. Accuracy of survey.
f. Description of special points.

PART 1B –HYDROLOGICAL INVESTIGATIONS

TABLE OF CONTENTS

Part

1B

Hydrological Investigations

1. PURPOSE OF GUIDELINES

Part 1B of the *Design Guidelines for Headworks of Hydropower Projects* establishes procedural guidelines for hydrological investigations performed in support of design of headworks for run-of-river hydropower projects. The guidelines are intended to support, direct and confine hydrologists in conducting hydrological investigations so that they can adopt a uniform and consistent approach in such investigations, taking fully into consideration the unique hydro-meteorological characteristics of Nepal and the availability of hydro-meteorological data, both in terms of quantity and quality, in Nepal.

2. SCOPE OF GUIDELINES

The guidelines cover the use of proper hydrological investigation techniques and methods of analysis for hydrological studies of headworks area for run-of-river hydropower projects. They outline the hydrologic study and analysis based on different combinations of data availability at the proposed headworks site, either upstream or downstream in the same catchment, and at hydro-meteorologically similar catchments.

3. TERMINOLOGY

Terms used in these guidelines are defined below:

Base flow	Stream flow rate occurring during recession of a hydrograph.
Basin	Surface area within a given drainage system.
Basin characteristics	Physical characteristics of a drainage basin that control its average hydrologic response in terms of runoff.
Basin slope	Rate of change of basin elevation with respect to the distance along the principal flow path.
Channel length	Distance measured along the main channel from the watershed outlet to the end of the channel.

Channel slope	Gradient measured by drop in elevation over channel distance.
Consistency	Status of agreement or compatibility among hydrologic data if no unusual changes are present in the data.
Daily flow records	Record of average daily flows at a stream gauge.
Envelope curve	Smooth curve covering all peak values of events plotted against other factors, such as area or time.
Flood	Runoff event that causes a river to rise above normal non-damaging limits.
Flood hydrograph	Record of continuous stream flow versus time for a given flood at a selected location on a stream.
Flood peak	Highest flow discharge attained during the passage of a flood wave
Gauged river basin	Basin where stream flows data are recorded at stations within the basin and are sufficient in quantity and quality to provide confidence in development of a hydrograph at the drainage basin outlet.
High-water mark	Mark, which identifies the maximum stage that occurred at a particular location during a historical flood.
Homogeneous data	Hydrological data that come from the same phenomena during the same period.
Long term data*	Hydro-meteorological data covering a period of 30 years or more.
Probable maximum flood	Flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study.
Probable maximum precipitation	Greatest depth of precipitation theoretically for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of year.
Short term data	Hydro-meteorological data covering a period of less than 30 years.
Time of concentration	Time for water to flow from the remotest part of the basin to its outlet point.
Ungauged river basin	Basin for which available hydrologic data, recorded at stations within the basin, are insufficient in quantity and quality to provide confidence in development of an inflow hydrograph, or a basin for which input and output measurements necessary for calibration are not available.

4. OBJECTIVE OF INVESTIGATIONS

Hydrological investigations performed for design of headworks of run-of-river hydropower projects shall generate hydrological estimates and predictions required for hydraulic design of headworks structures. To this end, these investigations shall aim at the following:

a. Providing input for the selection of return period for inflow design flood, construction diversion flood and low flows.

b. Predicting mean monthly flows as well as low flows and floods of different return periods.

c. Developing flow duration curves, mean monthly hydrographs, rating curves and water surface profiles at the headworks.

* Definition based on present condition of hydro-meteorological data availability in Nepal, to be revised in future.

5. SCOPE OF INVESTIGATIONS

To achieve the objectives listed in Section 4, the activities enumerated below shall normally be performed:

a. Establishment of a reliable hydro-meteorological database.
b. Extension of hydro-meteorological data.
c. Determination of basin and channel characteristics.
d. Study of snow and glaciers.
e. Analysis for long-term hydrology at the proposed site.
f. Analysis for low flow hydrology at the proposed site.
g. Analysis for flood hydrology, including Glacier Lake Outburst Flood (GLOF) and Cloudburst Flood (CLOF), at the proposed site.
h. Preparation of rating curves and water surface profiles at headworks.

For a given headworks, the need and scope of the above-listed activities shall be governed by the planning considerations discussed in Section 6. The activities thus planned shall be conducted using the procedures described in the subsequent sections.

6. INVESTIGATION PLANNING

Before being initiated, hydrological investigations shall be properly planned to define their scope, extent and nature. The planning shall be accomplished primarily through desk studies and site visits conducted jointly by the headworks design team and the investigation team.

6.1 Planning Considerations

Hydrological investigations shall be planned considering the data requirements for hydraulic design, data availability, existing hydro-meteorological network, factors influencing the river hydrology and the project development schedule.

6.1.1 Data Requirements

Hydrological investigations shall be geared towards generating appropriate hydrological data needed specifically for the design of headworks of run-of-river schemes, viz. data on long-term flows, low flows and flood flows. In generating these data, the investigations shall also take into account the type of run-of-river scheme proposed, i.e. simple or pondage run-of-river scheme.

Hydrological investigations shall be conducted in sufficient detail so that the data generated from them are well-founded, realistic and reliable. The data shall be worthy of being used with sufficient confidence in the design of the headworks, without introducing unwanted risks or unnecessary conservatism in it.

6.1.2 Data Availability

The hydrological investigation plans shall be prepared taking into account the nature of the hydro-meteorological data available for the river basin under consideration. In particular, the plans shall be evolved based on the type of data available, i.e. hydrology or meteorology, and their length of record, i.e. whether long term or short term.

Owing to its varying topography and altitude, Nepal experiences large variations in hydro-meteorology, both in space and time. However, the hydro-meteorological database presently available for most of the river basins in Nepal in very poor, with many river basins being ungauged. Even at most river basins that are gauged, the lengths of records available are generally inadequate by World Meteorological Organization standards.

6.1.3 Factors Influencing River Hydrology

Plans for hydrological investigations shall be drawn up considering the natural elements that influence the flows in Nepali rivers. These elements generally include snowmelt, monsoon rains and, occasionally, GLOF and CLOF.

Hydrological investigation plans for most rivers shall be based on the relative importance of snowmelt and monsoon rains on the river flows. In this context, it is worth noting that flows in most Nepali rivers are contributed to by snowmelt and monsoon rains, the extent of contribution depending on the altitude of the river basin. Rivers of major basins receive flow contributions from snowmelt and monsoon rains, with the contribution from the latter being substantial during monsoons and often resulting in floods. However, the effect of snowmelt becomes insignificant as the river basins move from the north to the south, so much so that basins entirely below 3,000 m do not experience any significant contribution from snowmelt.

For glacier-fed rivers, hydrological investigations shall generally include studies for GLOF. In particular, such studies shall be performed for rivers that originate from glacial lakes that have been identified as having the potential for bursting, e.g. Dig Tsho, Imja, Lower Barun, Tsho Rolpa, and Thulagi.

River originating from the Himalayan regions shall also be investigated for CLOF potential. Considering that CLOF events occur randomly and cannot be predicted precisely, CLOF studies shall be conducted for all rivers that have witnessed, or are prone to, CLOF events.

6.1.4 Schedule

To ensure that headworks designs are based on adequate and reliable hydrological data, flow measurements shall be commenced at the headworks site at an early stage of headworks development, preferably the pre-feasibility or feasibility stage of the project. Collection of data from gauging stations established for this purpose shall be planned to be continued during the design phase as well.

6.2 Planning Data and Information

Hydrological investigations for headworks design shall essentially be planned based on the available data and information on the drainage area characteristics as well as the hydrology and meteorology of the river basin. Such data and information shall be obtained from the sources indicated in the following sections.

6.2.1 Drainage Area Characteristics

Information on the physiographic characteristics of the drainage area, such as the latitude, altitude, area and shape of the basin, length of main channel, gradient of channel and basin, etc., shall be derived from 1:25,000 and 1:50,000 topographic maps published by the Survey Department, GoN. Similarly, information on geological formations, lithology and ground water table of the basin shall be obtained from regional geological maps published by the Department of Mines and Geology, GoN, and engineering geological maps prepared by the Department of Irrigation, GoN, and other agencies involved with hydropower development in Nepal.

6.2.2 Hydrological and Meteorological Data

Hydro-meteorological data from the hydrometric and meteorological stations existing in and around the river basin shall be obtained from publications of the Department of Hydrology and Meteorology (DHM), GoN. These data are computerized and can be obtained from the DHM on demand.

In addition, hydro-meteorological data may be obtained from reports on hydropower or irrigation projects studied in the river basin under consideration. Such reports are available for projects studied up to different levels by the Nepal Electricity Authority, DoED, Water and Energy Commission, Department of Irrigation, the erstwhile Electricity Department and private hydropower developers. Hydro-meteorological data may also be available at hydropower projects under operation in the river basin. However, any useful data from such sources shall be used after proper scrutiny.

6.2.3 Information on GLOF Potential

Information on the presence of glaciers and glacial lakes and the occurrences of GLOF in river valleys shall be gathered through study of aerial photographs and satellite images of the upper reaches of the river. Potentially hazardous or unstable glacial lakes shall be identified through regular baseline monitoring of the lakes using remote sensing or geographical information systems. The information thus collected may be verified with the inventory of glacial lakes in Nepal, including those with GLOF potential, prepared by the DHM and the International Center for Integrated Mountain Development (ICIMOD).

6.2.4 Information from Site Visit

During site visits, information on high flood levels at the proposed headworks site may be gathered from a study of flood marks. Information on upstream and downstream water uses and water rights shall also be collected during such visits.

7. ESTABLISHMENT OF RELIABLE HYDRO-METEOROLOGICAL DATABASE

Before hydrological analyses are initiated, a reliable hydro-meteorological database for the river basin shall be established to ensure generation of good quality hydrological estimates and predictions. For this purpose, the river basin shall be classified according to availability of hydro-meteorological data and their length of record in the river basin as well as at hydro-meteorologically similar catchments (HSC). Thereafter, data from the river basin and the HSC shall be collected, generated, compiled, suitably adjusted and checked for internal and external consistency.

7.1 Classification of Basins

Based on the possible combinations of hydrological and meteorological data availability at proposed site and at HSC, river basins shall be classified into two categories, viz.

a. Gauged river basins (GRB).

b. Ungauged river basins (URB).

Depending on the length of available stream flow record and rainfall record, these basins shall further be classified into seven categories listed in Table 1 and Table 2.

Table 1: Classification of gauged river basins (GRBs)

Basin	Data availability
G1	Long-term data on hydrology and long-term data on meteorology at proposed site
G2	Long-term data on hydrology at proposed site
G3	Short-term data on hydrology at proposed site and long-term data on hydrology at HSC
G4	Short-term data on hydrology and long-term data on meteorology at proposed site
G5	Short-term data on hydrology at proposed site and long-term data on meteorology with short-term data on hydrology at HSC
G6	Short-term data on hydrology at proposed site and long-term data on meteorology at HSC
G7	Short-term data on hydrology at proposed site

Table 2: Classification of ungauged river basins (URBs)

Basin	Data availability
U1	Long-term data on meteorology at proposed site and long-term data on hydrology at HSC
U2	Long-term data on hydrology at HSC
U3	Long-term data on meteorology at proposed site and short-term data on hydrology at HSC
U4	Long-term data on meteorology at proposed site
U5	Short-term data on meteorology at proposed site and long-term data on meteorology at HSC
U6	Short-term data on meteorology at HSC
U7	No data at all

7.2 Identification and Verification of HSC

For sites where hydro-meteorological data is either not available or inadequate, an HSC with hydro-meteorological data shall be identified so that the data for the HSC can be transposed to the proposed site. Generally, data from an HSC shall be transposed for the prediction of long-term monthly flows. Such transposition may also be used for prediction of low flows by analyzing and regionalizing DHM data. For prediction of floods, data transposition from an HSC shall not be performed if a series of instantaneous flood peaks is to be used for this purpose; however, the transposition may be carried out if yearly maximum series are used.

To permit data transposition, the HSC shall have the same runoff response as the catchment under study, for which the hydro-meteorological parameters and basin characteristics of the former need to be identical to those of the latter. To ensure this, the hydro-meteorological parameters and basin characteristics of the two catchments shall be properly compared and verified by homogeneity tests and sensitivity analyses. As homogeneity tests appear difficult in the context of Nepal, the parameters listed in Table 3 shall be compared, and the similarity between the identified HSC and the concerned basin shall be established logically with proper justification.

Table 3: Parameters for identification of hydro-meteorologically similar catchments

Basin and channel characteristics	• Basin area (total area and area below 5,000 m) • Basin shape factor • Drainage density • Time of concentration • Length of main channel • Slope of channel and basin
Meteorological characteristics	• Average annual rainfall • Monsoon wetness index • Climate
Topographical and geographical characteristics	• Short distance (neighboring basins) • Latitude and altitude • Facing (north, south) • Soil type (rock, boulder, sand, clay) • Vegetation (forest, agricultural land, barren land) • Storage (lakes, reservoirs, ponds) • Lithology and ground water table, springs, etc.

As all parameters listed in Table 3 are not likely to be similar or identical, catchments may be considered similar if some sensitive parameters are more or less identical or similar. For long-term monthly flows, the sensitive parameter shall include basin area, drainage density, annual and monsoon rainfall, altitude, facing, storage effect, etc. Likewise, for prediction of low flows, the sensitive parameters shall include lithology, ground water table, springs and basin area above 5,000 m (area of snow melting). Where prediction of floods is performed using yearly maximum series, the most sensitive parameters shall be the monsoon wetness index, basin area below 5,000 m, time of concentration, drainage density, basin shape factor, etc.

7.3 Collection, Generation and Compilation of Data

After completing the foregoing procedures, all hydro-meteorological data available in the concerned catchment and identified HSCs, along with data gathered from gauging stations established at the proposed headworks site, shall be collected. The collected data shall be described and documented with inventories in the form of bar diagrams, indicating the source, location, altitude, drainage area (where appropriate), period of availability in respect of all stations within the area of interest and the surrounding region. The source of the collected data shall also be mentioned.

Following their collection, the flow data shall be checked with the rainfall data, and long series of flow data shall be generated. If necessary, regional analyses shall be carried out.

After data collection and generation, suitable data series shall be compiled for the required analyses. These series shall include the annual maximum flow series for flood frequency analysis, the annual minimum flow series for low flow analysis and the mean monthly flow series for generation of flow duration curves and monthly hydrograph. In the case of annual series, a good theoretical basis exists for extrapolation.

The consistency, reliability and adequacy of selected data shall be checked. The basic data shall first be screened and adjusted to remove, as far as possible, any nonconformity that may exist. Data shall be prepared for different analysis.

While preparing data for analysis, the following considerations shall be borne in mind:

- Effects of manmade changes in the flow regime shall be investigated and adjusted as required.
- For small watersheds, a distinction shall be made between the daily maxima and the instantaneous or momentary flood peaks.
- As changes in stage-discharge relationships make stage records non-homogenous and unsuitable for frequency analysis, it shall be preferable to work with discharges. In case stage frequencies are required, the results shall be referred to the most recent rating.

7.4 Collection and Use of Stream Flow Data

In order to draw meaningful conclusions on the stream flow, observed flow data is necessary. Gauging the flow in a stream can collect this data. The objective of stream gauging is to estimate the discharge in the stream. Such measurement of discharge is made once in a day, which is presumed to be the mean value for that day. The gauging record for daily mean discharges yields daily flow data.

Stream flow data can be used to study the significant properties of flow on one hand and to establish a correlation of the flow with the rainfall on the other hand. Among the significant properties of flow, the maximum and minimum discharges to be anticipated during the life of the power project, the seasonal and monthly variations in discharge, the total volume of water available in a season or in a year and its dependability are required to be known.

The analysis for the maximum discharge and the minimum discharge is known respectively as flood analysis and low flow analysis. In this analysis, it is required to find out not only the extreme values (highest probable flood or lowest probable flow) but also the actual duration of such conditions and a precise day-to-day sequential variation that can be anticipated.

In the volumetric estimation of runoff, the actual sequence of runoff is of secondary importance but the dependability of such estimation has to be analyzed. This can be termed as runoff analysis.

Stream flow data is usually available in tabular form. At the time of analysis, it is first represented graphically for which various types of plots are used such as hydrograph, mass curve, flow duration curve etc.

The accuracy of any hydrological prediction of rainfall or river flow is mainly dependent on the length and reliability of records. Ideally, therefore, long-term rainfall records are needed from the project area, the main river catchment and the cross-drainage catchments together with discharge records in the main river and, if possible, the major cross drainage channels. Existence of long-term rainfall records enables reasonable, direct estimates of reliable rainfall and storm runoff. If sufficiently long stream flow records are available, these can be correlated with the rainfall data and estimation of flows would be more accurate.

If no catchment rainfall records exist and if the records (both rainfall and stream flow) within the project area are of short duration and dubious validity, both rainfall and run-off have to be assessed indirectly. If rainfall records are available from other sites of Hydro-meteorologically Similar Catchment (HSC), these could be used as guides but careful consideration of reliability of data, latitude, altitude, geographical location, and degree of exposure, etc. has to be included. Again, to relate the chosen rainfall to river flow, it is necessary to make indirect assessments and comparisons with records from rivers elsewhere with similar terrain, vegetation cover and climate.

7.5 Measurement of Stream Flow

Stream flow may be measured at the headworks site using one or a combination of the following methods:

a. Hydrometric (velocity-area) methods.
b. Hydraulic methods.
c. Dilution method.
d. Volumetric method.

7.5.1 Hydrometric Method (Velocity-Area Method)

Hydrometric measurements shall generally be conducted using the current meter. Where use of the current meter is not possible due to unsuitable velocities or depths of flow, presence of materials in suspension or limited availability of time for flow measurements, floats shall be adopted for measuring the stream flow; however, float measurements results in lesser accuracy.

7.5.1.1 Current Meter Measurements

Using a current meter, the total discharge Q through the selected river cross-section shall be obtained as

Eq. 1

$$Q = \sum_i A_i V_i$$

where A_i are partial cross-sectional area of the river at the selected location and V_i are the corresponding mean velocities of flow measured normal to the partial areas.

In shallow waters, velocity may be observed at one point, at 0.5 or 0.6 of the effective depth measured from the bottom, and a coefficient shall normally be applied to convert the observed velocity to the mean velocity. In deeper waters, the velocity measurements could include two observations at 0.2 and 0.8 of the effective depth, three observations at 0.15, 0.5 and 0.85 of the effective depth or six observations at 0.2, 0.4, 0.6 and 0.8 of the effective depth, and at points close to the water surface and river bed. For the two and three point methods, the average of the observed velocities may be used as the mean in the vertical. For the six-point method, the mean velocity V_m shall be computed as

Eq. 2 $$V_m = 0.1(V_s + 2V_{0.2} + 2V_{0.4} + 2V_{0.6} + 2V_{0.8} + V_b)$$

where V_s and V_b are velocities measured near the water surface and river bed, respectively, while $V_{0.2}$, $V_{0.4}$, $V_{0.6}$ and $V_{0.8}$ are velocities measured at 0.2, 0.4, 0.6 and 0.8 of the effective depth, respectively.

7.5.1.2 Float Measurements

In principle, any drifting object may serve as a float. Different types of floats, such as the surface float, depth float, double float, floating rod and depth-integrating float, etc., may be used for the measurement; however, the use of surface floats may generally suffice.

For flow measurements, the velocity of flow shall be estimated by recording the time taken by floats to travel between two chosen cross-sections of the river. Generally, a minimum of 15 to 25 floats, uniformly distributed over the river width, shall be used for this purpose. The floats shall be released sufficiently upstream of the river cross-section chosen for initiating velocity measurement so that they attain a constant velocity by the time they reach this cross-section. As some floats may touch the river bottom, repetitive float measurements shall be taken.

At each site, the mean velocity of river flow shall be computed from the float-measured velocities by applying a coefficient determined by calibrating the flow measurements from the float method against those from the current meter method. Where such measurements are not available, an adjustment factor, *F*, which is a function of the ratio of the immersed depth of float to the depth of water (*R*), may be used for rough estimation (Table 4).

Table 4: Adjustment factors for float-measured velocity

R	0.10 or less	0.25	0.50	0.75	0.95
F	0.86	0.88	0.90	0.94	0.98

7.5.2 Hydraulic Method (Slope-Area Method)

Hydraulic methods shall be used to measure flows indirectly when direct flow measurement is not possible, e.g. during floods because of excessive rate of change of discharge, excessive velocities, debris, depths or widths of flow, inaccessibility of measuring structures, etc. They shall generally be used at flow-measuring structures such as dams, weirs, sluices, highway embankments, etc. or through selected reaches of river channels, culverts, bridge openings, etc. River channels selected for such measurements shall have uniformity or uniform variation in hydraulic properties. Normally, contracting river reaches shall be preferred over expanding reaches.

Hydraulic methods shall consist of measuring the fall between the upper and lower sections of the water surface profile and computing the flow velocity through hydraulic formulae appropriate to the type of waterway chosen. Application of these formulae shall involve assessment or measurement of the following factors:

a. Physical characteristics of the channel, geometry of the channel within and adjacent to the reach used and the boundary conditions.
b. Water-surface elevations at the time of peak stage to define the upper limits of the cross-sectional areas and the difference in elevation between two significant points.
c. Hydraulic factors, such as roughness coefficients, based on physical characteristics.

7.5.3 Dilution Method

Using this method, the discharge shall be obtained by measuring the concentration of water injected with tracer solution passing through a sampling point. The sampling shall be done after complete mixing of the tracer solution has taken place.

7.5.4 Volumetric Method

The volumetric method shall be used for measuring extremely small flows, such as those from springs or in tiny rivulets. The flow shall be obtained directly by measuring the volume of water flowing over a certain period.

7.6 Data Processing

Normally, the basic observed data shall be processed before or immediately after their publication to organize them in forms that are useful to analysts. The data processing shall be performed as discussed in the following sections.

7.6.1 Stream Flow Rating Curves and Their Extrapolation

Stream flow data shall be processed and presented in graphical form as rating curves or in tabular form as rating tables. In addition, a continuous record of flow at a gauging station may be computed from a record of the stage and a stage-discharge relation (rating curve) through extrapolation. The end product of this computation shall be a tabulation of the mean discharge for each day, month and year. The tabulation shall also include a list of the instantaneous peak discharges.

7.6.2 Precipitation

Precipitation data may be presented in the form of average depth of precipitation over a specific area on a storm, seasonal or annual basis. As run-of-river hydropower projects are generally planned in hilly or mountainous areas, the Thiessen polygon method and the isohyetal method shall be used for averaging the precipitation over the catchment area.

7.6.3 Procedures for Data Processing

The following procedures shall be adopted to process hydrological and meteorological data for hydrological investigations:

a. Notes on the method of measurement, standards followed, instruments used, frequency of observations, history of the station, shifts in location of the station and shifts in ratings shall be prepared.

b. If the given discharges are observed discharges or are computed from observed stages using stage-discharge ratings, the development of stage-discharge rating curves at the gauge-discharge site, including the extrapolation involved, shall be discussed.

c. The need for filling up small data gaps by gap-filling techniques shall be stated. Gap-filling techniques may include:
   - Random choice from values observed for the period in question.
   - Interpolation from adjoining values by plotting a smooth hydrograph (for runoff alone).
   - Using the average proportion with normal for adjoining stations.
   - Double-mass curve technique.
   - Correlation with adjoining stations either of the same hydrologic element or of a different hydrologic element.
   - Autocorrelation with earlier periods at the same station.

d. Gap filling for annual maximum discharge may not be necessary if the gap itself is not caused by any hydrologic reasons which are liable to introduce bias in the incomplete sample (e.g. washing away of the gauge due to a very high flood. In this case, the whole series shall be treated as one, i.e. the gap shall be ignored.

7.7 Adjustment of Flow Record

As their use in hydrological studies is normally necessary and convenient, natural or virgin flows shall be computed from the observed flows by making corrections for the effects of human activities. While doing so, the procedures shall be observed:

a. Withdrawals out of the system shall be considered in checking the overall water balance.
b. The need for adjustment to "natural" flows and the manner in which this is done shall be discussed and supported with withdrawal data, reservoir data or data on irrigation statistics. Where adjustments due to upstream storage are made, it shall be ensured that storage changes and evaporation losses are properly accounted for.
c. Adjustments made in low flows shall be stated. Discussion shall be made if, apart from adding diverted flows to the observed flows, return flows have been subtracted.
d. When long historical runoff data shows a trend attributable to change in land use, extrapolation of the trend and correction of the historical series to make it compatible with present and future conditions shall be performed.
e. When effects on flood flows due to existing projects are considered appreciable, the adjustment of flood flows for the effect of storage shall be made by calculating the change introduced by the reservoir at the control point and correcting for it. This may involve reservoir and channel routing.

7.8 Internal Consistency of Data

To check the internal consistency of data, the stage-discharge relation of observed data shall be studied. If the relation is not stationary, analyses shall be performed to find whether there are physical reasons for this phenomenon. For points showing large deviations, investigation shall be carried out to detect likely errors. These studies shall be performed for all stage-discharge sites used.

7.9 External Consistency of Data

In order to verify external consistency of observed data, the following checks shall normally be performed:

- Consistency between rainfall and runoff shall be checked by comparing the annual monsoon months' rainfalls in the drainage area and the runoff at the gauging site. This may be done for the full length of the concurrent record at all sites used in the study.
- Consistency of runoff shall be checked by comparing the average annual specific flow (expressed in liters/s/km^2) with the corresponding figures for adjoining and similar basins. When there is more than one site on a river, the figures of the average annual specific flow for sub-areas shall be calculated and compared. This study shall be conducted for all discharge sites in the region, including the central network sites nearby.
- Consistency of short period runoffs shall be checked by comparing them with concurrent short period runoffs at adjoining sites. For this, hydrographs of daily flows shall be superimposed with discharges plotted on log scale (semi-log paper) for visual comparison. When inconsistency is seen, errors shall be checked.
- For all stations with known or suspected changes in location or exposure conditions, the quality of rainfall data shall be checked by double-mass curve techniques. If a kink is observed, the data shall be made consistent either to the old or the new conditions.
- For runoff data, a similar technique of checking by the double-mass curve may be used when a change in location or method of observation is known or suspected.

8. EXTENSION OF HYDRO-METEOROLOGICAL DATA

As the long-term data required for needed design of hydropower projects are not adequately available in Nepal, short-term hydro-meteorological data may, where required, be extended using the following long-term records:

a. Long-term stream flow data at HSC.
b. Long-term precipitation record.

8.1 Extension with Long-term Stream Flow Data at HSC

If stream flow data for a sufficiently long period are available at an HSC or at a downstream location on the same river, these stream flow record may be extended using the following methods:

a. Double mass curve method.
b. Basin area ratio method.
c. Regression analysis method.
d. Index station method.
e. Lanbein's log-deviation method.
f. Mean ratio method.

8.1.1 Double-Mass Curve Method

In this method, the cumulative stream flow of the base station (proposed site under study) and the index station shall be plotted on a graph, and the relation between stream flows at the two stations shall be obtained from the slope of the double-mass curve thus generated. While using this method, note should be taken of the limitation of that this method that it yields a constant slope regardless of variation in size of yearly (or monthly) increments, even though the flow records of the two streams may not necessarily correlate with each other as straight lines.

8.1.2 Basin Area Ratio Method

If two basins are hydro-meteorologically similar, data extension may accomplished simply by multiplying the available long-term data at the HSC with the ratio of the basin areas of the base station (proposed site under study) and the index (HSC) station. In the context of Nepal, more accurate results may be obtained using Dicken's flood formula

Eq. 3

$$Q_b = Q_i \left(\frac{A_b}{A_i} \right)^{3/4}$$

where Q_b and Q_i are the discharges at the base and index stations, respectively, and A_b and A_i are the corresponding basin areas.

8.1.3 Regression Analysis Method

Using regression analysis, the association between the discharge at the base station, Q_b, and that at the index station, Q_i, may be determined by fitting the regression line

Eq. 4

$$Q_b = a + bQ_i$$

where a and b are regression coefficients which can be determined by standard formula.

8.1.4 Index Station Method

In this method, stream flow data extension shall be achieved using duration curves. For this, the values of the hydrologic event shall be arranged in descending order of magnitude, and the percent of time that each magnitude is equaled or exceeded shall be computed. The duration curve shall then be prepared by plotting the magnitudes as ordinates and the corresponding percentage of time as the abscissa.

8.1.5 Langbein's Log-Deviation Method

Using this method, correlations between the base and index stations shall be made in terms of the logarithms of discharges to remove the skewness inherent in stream flow data. The correlation shall usually be made between the 10-daily mean discharges or the monthly mean discharges; however, flood peaks, daily means or annual means could also be used for this purpose.

For correlation, the regression equation shall be of the form

Eq. 5 $$Y = bX$$

where Y and X are log-deviations for discharges at the base station and the index station, respectively, and

Eq. 6 $$b = \frac{\sum XY}{\sum X^2}$$

The correlation coefficient for the regression analysis shall be computed as

Eq. 7 $$r = \frac{\sum XY}{\sqrt{\sum X^2 \sum Y^2}}$$

For data extension, Y shall be computed for the available discharge at the base station by deducting the mean of the logarithms of discharges from the logarithm of discharges. X shall similarly be computed for the concurrent discharge data at the index station. If the value of r computed from Eq. 7 using these values of X and Y is satisfactory, i.e. $r > 0.6$, b shall be computed from Eq. 6, and the log-deviation Y for the base station for the extended period shall be estimated from the log-deviation X of the index station using Eq. 5. After this, logarithms of discharges at the base station shall be estimated by adding the mean of logarithms of the base station to the computed Y. Antilogarithms of the values estimated in the foregoing step shall give the required data at the proposed site extended from the HSC.

8.1.6 Mean Ratio Method

This method consists of computing the mean ratio, r_m, for the base and index stations as

Eq. 8 $$r_m = \frac{Q_{bm}}{Q_{im}}$$

where Q_{bm} is the mean of the short-term record at the base station and Q_{im} is the long-term record for the same period at the index station of the HSC. The remaining records at base station shall then be calculated by multiplying each record of the index station of the HSC with r_m.

8.2 Extension with Long-term Precipitation Record

Extension of short-term data using long-term precipitation records may be performed for the following cases that generally arise in practice:

Case I	Long-term precipitation records and stream flow data for a few years are available at the site.
Case II	Long-term precipitation record is available at the site and stream flow data for a

	few years or concurrent rainfall-runoff data are available at an HSC.
Case III	Only precipitation record is available at the site.
Case IV	No data are available.

8.2.1 Extension for Case I

When long-term precipitation records and a few years' stream flow data are available at the site, a statistical correlation between the observed precipitation and the stream flow shall be established and plotted on a log-log graph. If the relationship is not a straight line, it shall be suitably extended to find out the stream flow corresponding to the weighted precipitation of each year.

8.2.2 Extension for Case II

When long-term precipitation record is available at the site and a few years' stream flow data or concurrent rainfall-runoff data are available at an HSC, the rainfall-runoff correlation shall be established as for Case I (refer Section 8.2.1) for the HSC. The runoff series at proposed site shall then be worked out by feeding the long-term rainfall data of the base station in the correlation equation thus established.

8.2.3 Extension for Case III

If only precipitation records are available at the site, empirical formulae may be used to compute runoff (Mutreja, 1995). For Nepal, the empirical formulae discussed below may be used with extreme caution.

Khosla's Formula

Using Khosla's formula, the runoff R in cm shall be computed as

Eq. 9
$$R = P - \frac{T}{3.74}$$

where P is the precipitation in cm and T is the mean temperature in °C. Similarly, the monthly runoff R_m in mm shall be obtained from the relation

Eq. 10
$$R_m = P_m - L_m$$

where P_m is the monthly precipitation in mm and L_m represents the monthly losses in mm. L_m shall be estimated from the mean monthly temperature T_m. For $T_m > 4.5$ °C,

Eq. 11
$$L_m = 5T_m$$

For $T_m < 4.5$ °C, L_m shall be adopted from Table 5.

Table 5: Relation between mean monthly temperature and monthly losses

T_m (°C)	4.5	-1	-7	-12	-18
L_m (mm)	21	18	15	12.5	10

(Source: Mutreja, 1995)

UPIRI Formulae

Among the several statistical correlations between runoff and precipitation developed by the Uttar Pradesh Irrigation Research Institute for Himalayan rivers in Uttar Pradesh, India, the following formula derived for the Ganga basin at Hardwar may be used to estimate runoff for Nepali rivers:

Eq. 12
$$R = 5.45P^{0.6}$$

where R is the runoff in cm and P is the precipitation in cm.

Likewise, the following relation developed for the Sharda basin at Banbasa may be used for this purpose:

Eq. 13
$$R = 2.7P^{0.8}$$

Extreme caution shall be exercised in using Eq. 12 and Eq. 13.

ICAR Formula

The runoff from small watersheds, having an area up to 100 sq. km, may be estimated from the following formula developed by the Indian Council of Agricultural Research (Mutreja, 1995):

Eq. 14
$$Q = \frac{1.511P^{1.44}}{T_m^{1.34} A^{0.0613}}$$

where Q is the annual runoff in cm, P is the annual precipitation in cm, T_m is the mean annual temperature in °C and A is the watershed area in sq. km.

8.2.4 Extension for Case IV

Where no data is available, the stream flow (generally flood) at the proposed site may be estimated by making a regional flood-frequency analysis in the following two steps:

a. Plotting a curve between the catchment area and the mean flood.

b. Plotting a curve between return periods and the ratio of flood to mean flood.

9. DETERMINATION OF BASIN AND CHANNEL CHARACTERISTICS

For hydrological analyses, requisite basin and channel characteristics shall be estimated. The basin characteristics shall consist of the basin area, slope, shape and average width while the channel characteristics shall include the channel length, slope, drainage density and time of concentration.

9.1 Basin Area

Basin area shall be estimated by a number of methods, and a suitable value for it shall be adopted after comparing the areas estimated from the different methods. An average of the different estimated values may also be adopted.

For estimating the basin area, the watershed divide line shall be marked on a topographic map considering the physical law of surface runoff that water flows perpendicularly to contour lines. This divide line shall be thoroughly checked to avoid mistakes leading to overlap of adjacent basins. The delineated basin area shall then be accurately measured on the map by a planimeter or by tracing the catchment boundary on a transparent graph paper and counting square grids and triangles.

9.2 Basin Slope

To estimate the basin slope, the principal flow path shall be delineated on a topographic map. The basin slope shall then be computed as the ration of the difference in elevation between the end points of the principal flow path and the length of the flow path or channel length.

9.3 Basin Shape

The basin shape may be reflected using different watershed parameters. This may be done using a coefficient defined as L^2/A, where L is the total length of the channel and A is the basin area.

9.4 Average Width of Basin

The average width of the basin may be computed as the ratio of the basin area A and the channel length L.

9.5 Channel Length

Normally, the channel length shall be determined by field survey of the streamline or the centerline of the stream. It may be measured from a map using dividers or threads; however, to account for the fact that streams are simplified when drawn on maps, the measured length shall be multiplied by a tortuosity coefficient 1.01 to 1.15. Alternatively, the channel length L in km may be estimated from the following empirical relation (Jha, 1996):

Eq. 15 $$L = 2.82\sqrt{A}$$

where A is the basin area in sq. km.

9.6 Channel Slope

The channel slope S shall normally be estimated from the relation

Eq. 16 $$S = \frac{\Delta E}{L}$$

where ΔE is the difference in elevation between the points defining the upper and lower ends of the channel and L is the length of the channel between these two points. An approximation to the slope may also be computed from the equation

Eq. 17 $$S = \frac{(h_{\max} - h_{\min})}{\sqrt{A}}$$

where $h_{\max}$ and $h_{\min}$ are the highest and lowest points, respectively, in the basin and A is the basin area.

9.7 Drainage Density

The drainage density shall be estimated as the ratio of the total length of streams within a watershed to the total area of the basin.

9.8 Time of Concentration

The time of concentration t_c, in hours, may be estimated by using Kirpich's formula

Eq. 18 $$t_c = 0.00032 L^{0.77} S^{-0.385}$$

where L is the maximum length of travel of water in m and S is the slope equal to H/L, H being the difference in elevation between the remotest point of the basin and the outlet in m.

Alternatively, t_c may be estimated using Mockus's formula given by

Eq. 19 $$t_c = \frac{t_p}{0.6}$$

and

Eq. 20 $$P_R = \sqrt{t_c} + 0.6 t_c$$

where t_p is the lag time, i.e. the time from the center of mass of excess rainfall to the peak runoff rate, in hours and P_R is time from the beginning of runoff to the time of peak runoff in hours.

10. STUDY OF SNOW AND GLACIERS

Study of snow and glaciers shall be conducted for rivers whose dominant source of flow is snow or a glacier. This study shall include measurement of snow and estimation of runoff from snowmelt.

10.1 Measurement of Snow

In regions where snow cover is not continuous, the depth and density of snow cover shall be measured to predict the overland flow resulting from snowmelt. This shall be achieved by means of surveyed snow courses, which are sections of the snow cover, whose depth shall be determined by means of gauges that are installed prior to any snowfall. The density of snow in a snow pack may be determined by boring a hole through or into the pack and measuring the amount of water obtained from the sample collected. Automatic devices for measuring the weight of the snow above a certain point in the ground may also be used for this purpose.

10.2 Methods for Estimation of Snowmelt Runoff

The runoff from snowmelt may be determined using various approaches. These may include the following:

a. Regression analysis.
b. Physical equations for basin snowmelt
c. Analysis of hydrograph recessions.
d. Hydrograph syntheses.

For open sites in ungauged basins, the following empirical equations may be used to predict the daily snowmelt M in cm:

Eq. 21 $$M = 0.03(9T_{mean} + 40)$$

Eq. 22 $$M = 0.02(9T_{\max} + 25)$$

where T_{mean} and $T_{\max}$ are the mean and maximum daily temperature, respectively, in °C. For forest sites in ungauged basins, M may be computed from the following relations:

Eq. 23 $$M = 0.025(9T_{mean})$$

Eq. 24 $$M = 0.02(9T_{\max} - 50)$$

11. FLOOD HYDROLOGY

Studies on flood hydrology shall be conducted to estimate the following parameters:

a. Floods of different return periods for selection of inflow design flood for the overflow section of the diversion structure.
b. Probable maximum flood.
c. GLOF and CLOF.

11.1 Flood Estimation for Different Return Periods

Depending upon data availability, floods of different return periods may be estimated using one or more of the following methods:

a. Flood frequency analysis (statistical and regional).
b. Regional methods.
c. Empirical formulae.
d. Enveloping curves.

If long-term maximum instantaneous flow data are available in a GRB, statistical frequency analysis shall be used to reliably estimate the design flood. In a URB with no data, regional methods, empirical formulae, envelope curves, etc. may be used methods for this purpose. The results from these analyses may be substantiated through *in situ* flood investigations and through the slope area method.

11.1.1 Flood Frequency Analysis

Flood frequency analysis shall be used to estimate the peak discharge of known frequency directly with the runoff (stream flow) data. This analysis shall preferably be used to estimate floods magnitudes for return periods less than the periods of the observed record where the estimation can be performed by interpolation. When floods approaching the limits of the record are to be estimated, the analysis shall be performed after thoroughly understanding of the exact nature of the population distribution from which this record is derived. Where extrapolation of records is necessary to estimate flood flows of large return periods, the extrapolation may be limited, say up to twice the length of record, to ensure accuracy of the estimate. Alternatively, extrapolation may be performed to estimate floods of large return periods, but they shall be accompanied with an estimate of the confidence limits to serve as a guide to the degrees of uncertainty involved in the estimates.

In Nepal, only a few sites have stream flow records covering a period of even 30 years, and these records generally contain few flood events. As such, the flood frequency analysis shall be performed to estimate floods with due caution.

11.1.1.1 Data Requirement

For flood frequency analyses, an annual series, formed from the maximum observed floods of each year, shall commonly be used, particularly where extrapolation of records is needed. Alternatively, a partial duration series, formed by listing all high flows above a particular base – usually the lowest flood of the annual series, may be used for the analyses. Use of this series shall be restricted to estimate floods within the range of observations.

Before being used to form the above-mentioned series, the basic stream flow data shall be screened and adjusted to remove, as far as possible, any nonconformity that may exist in the data. Such nonconformities could result from changes in factors influencing the character of each event, variations in measurement technique and size of observation, etc.

11.1.1.2 Methods of Analysis

Where sufficiently long and reliable records are available, frequencies (or probabilities) may be evaluated by fitting a smooth curve through a plot of the magnitudes of flood flows against the frequencies with which they have been equaled or exceeded. The curve may be assumed to be representative of future possibilities.

Where the length of record is not adequate for the above-mentioned curve fitting, two types of distributions, viz. logarithmic normal distribution and extreme value distribution, shall be employed to standardize the basis for curve fitting. Accordingly, the analysis shall be carried out using the following methods:

a. Curve fitting methods, graphical or mathematical.
b. Methods using frequency factors, such as Gumbel and Lognormal methods.

The curve fitting methods may use different formulae to determine the plotting positions of distribution; however, Weibull and California formulae may be preferred for this purpose. Methods based on frequency factors shall use a generalized equation, and a plot of the observed data may be made to see how closely the estimated frequency line fits the observed data.

11.1.2 Regional Flood Frequency Analysis

When the available data at a basin is too short to conduct frequency analysis, a regional analysis may be adopted. In this, a hydro-meteorologically homogeneous region from the statistical point of view shall be considered, available long-term data from neighboring basins shall be tested for homogeneity and a group of stations satisfying this test shall be identified. This group of stations shall be considered to constitute a region, and the entire station data of this region shall be pooled and analyzed as a group to find the frequency characteristics of the region.

Regional flood frequency analysis shall be conducted in two parts. The first part shall consist of developing a basic dimensionless frequency curve that represents the ratio of the flood of any frequency to the mean annual flood ($Q_T/Q_{2.33}$) versus the return period T, with the mean annual flood corresponding to a recurrence interval of 2.33 years. The second part shall consist of developing relations between topographic characteristics of the drainage area and mean annual flood i.e. A versus $Q_{2.33}$, to enable the prediction of $Q_{2.33}$ for an ungauged basin at any point within the region. The combination of the $Q_{2.33}$ of an ungauged basin with the basic frequency curve of homogeneous gauged basins in terms of $Q_{2.33}$ shall then be used to obtain an estimate of design flood for the required frequency at any location in the region.

11.1.3 Considerations for Flood Frequency Analysis

Flood frequency analysis shall be conducted considering the following issues:

- When adequate flow data (usually more than a 25-year record) are available and flood magnitudes of large return periods are the primary concern, the annual series data shall be plotted on both extreme value and lognormal probability papers. The plots shall be used to draw a tentative idea on whether a lognormal or extreme-value fit would be more appropriate, the criteria for appropriateness being the closeness to the linearity of a line joining the plotted points. Thereafter, the observed data shall be fitted to the selected distribution either by the frequency factor or the curve-fitting method.
- When available records are of shorter length (less than 25 years) and the object is to determine floods of smaller return periods (seldom exceeding 25 years), partial duration series shall be compiled and analyzed. The results may, however, be checked by analyzing the annual flood peaks by the Gumbel or lognormal method.
- When available data are too short or non-existent, regional flood frequency analyses can be carried out.
- Occasionally, the observed point or points may depart markedly from the computed frequency curve on probability paper. Such points that fall far outside the curve may be considered non-homogenous with the rest of the sample, and the curve shall be recomputed excluding them from the analysis.
- Wherever possible, a guide to the reliability of the curve shall be provided by estimating the confidence limits of the analysis. This shall be achieved by showing confidence bands on either side of the computed frequency curve on probability paper.

11.1.4 Regional Methods

In the absence of maximum instantaneous flow data at the proposed site, regional methods developed for ungauged locations in Nepal may be used for flood estimation. Such methods include the WECS/DHM method, the PCJ method and the MHSP method.

11.1.4.1 WECS/DHM Method

The WECS/DHM method (WECS/DHM, 1990) may be used for flood prediction for small hydropower projects located in ungauged basins of Nepal. Using this method, 2-year (median flood) and 100-year floods for both maximum daily and maximum instantaneous flood peaks shall be computed from regression equations of the form

Eq. 25 $$Q_{aby} = \alpha(A_{3000} + 1)^{\beta}$$

where $Qaby$ is the discharge in m^3/s, subscript a is either a daily or an instantaneous flood peak, subscript b is either a 2 year or a 100 year return period, A_{3000} is the catchment area below 3,000 m and α and β are coefficients. Thereafter, floods of other return periods shall be calculated simply by the plotting the 2 year and 100 year floods on log-normal probability paper, which results in a straight line, or using algebraic equations.

11.1.4.2 PCJ Method

Using the PCJ method (Jha, 1996), the maximum rainfall design discharge Q_p for the required exceedance probability p shall be computed using the equation

Eq. 26 $$Q_p = 16.67 a_p o_p \phi F k_F$$

where a_p is the maximum rainfall design intensity for p, o_p is the infiltration coefficient of the basin, ϕ is the reduction coefficient of maximum discharge that depends on the basin size and the power reduction indicator n (= 0.33 for Nepal), and k_F is a coefficient that reflects the unequal distribution of rainfall in different size of basin captured by one rain. In this equation,

Eq. 27 $$a_p = a_{hr} k_t$$

where a_{hr} is the hourly rainfall intensity for p and k_t is a reduction coefficient of hourly rainfall intensity that depends on the size of the catchment area.

11.1.4.3 MHSP Method

Based on the MHSP method (MHSP/CIWEC, 1997), the flood peak Q in m^3/s shall be computed using the relation

Eq. 28 $$Q = kA^b$$

where k and b are constants which depend on the return period T (Table 6).

Table 6: Constants for MHSP method

T (years)	5	20	50	100	1000	10000
k	7.4008	13.0848	17.6058	21.5181	39.9035	69.7807
b	0.7862	0.7535	0.738	0.7281	0.6969	0.6695

11.1.5 Rational Method

Flood flows may be estimated from rational formulae that take into account the intensity, distribution and duration of rainfall as well as the area, shape, slope, permeability and initial wetness of the basin. For small catchments up to 50 sq. km., the following formula could be used:

Eq. 29 $$Q_T = 0.278CIA$$

where Q_T is the maximum flood discharge in m^3/s for the required return period T, C is the runoff coefficient (Table 7), I is the mean intensity of rainfall in mm/hour for return period T and time of concentration t_c and A is the basin area in sq. km.

Table 7: Runoff coefficients for rational formula

Type of basin	***C***

Rocky and permeable	0.8 – 1.0
Slightly impermeable, bare	0.6 – 0.8
Cultivated or covered with vegetation	0.4 – 0.6
Cultivated absorbent soil	0.3 – 0.4
Sandy soil	0.2 – 0.3
Heavy forest	0.1 – 0.3

In the absence of data on rainfall intensity, I shall be estimated by

Eq. 30
$$I = \frac{KT^a}{(t_c + b)^n}$$

where K, a, b and n are constants to be defined for the particular site. For Nepal, their values may be assumed as those for Northern India, i.e. $K = 5.92$, $a = 0.162$, $b = 0.5$ and $n = 1.013$. For use in Eq. 30, the time of concentration t_c, in hours, shall be estimated by Kirpich formula

Eq. 31
$$t_c = 0.00032 L^{0.77} S^{-0.385}$$

where L is the maximum length of travel of water in m and S is the slope equal to H/L, H being the difference in elevation between the remotest point of the basin and the outlet in m.

11.1.6 Empirical Formulae

Empirical formulae shall be used only when a more accurate method for flood prediction cannot be applied because of lack of data. For flood prediction in ungauged basins of Nepal, the empirical formulae discussed in the following sections may be used with great caution and proper justification.

11.1.6.1 Modified Dicken's Method

Using Dicken's method, the T year flood discharge Q_T, in m^3/sec, shall be determined as

Eq. 32
$$Q_T = C_T A^{0.75}$$

where A is the total basin area in sq. km and C_T is the modified Dicken's constant proposed by the Irrigation Research Institute, Roorkee, India, based on frequency studies on Himalayan rivers. This constant shall be computed as

Eq. 33
$$C_T = 2.342 \log(0.6T) \log\left(\frac{1185}{p}\right) + 4$$

Eq. 34
$$p = 100 \frac{a+6}{A+a}$$

where a is perpetual snow area in sq. km. and T is the return period in years.

11.1.6.2 Snyder's Method

For an ungauged river, Snyder's method may be used for flood flow estimation by deriving a synthetic unit hydrograph based on known physical characteristics of the basin. In this method, the peak discharge Q_{PR}, in m^3/s, shall be computed as

Eq. 35 $$Q_{PR} = q_{PR} C_A A R$$

where q_{PR} is the peak discharge per square km of the drainage area due to 1 cm of effective rainfall for rainfall duration of t_r in $m^3/s/sq.$ km., C_A is an aerial reduction factor that accounts for the fact that the average rainfall intensity over a large area is smaller than that over a small area, R is the rainfall in cm for duration t_R derived from the 24-hour rainfall with the reduction for area. For use in Eq. 35, q_{PR} shall be computed from the relation

Eq. 36 $$q_{PR} = 2.78 \frac{C_P}{t_{PR}}$$

where C_P is a coefficient depending upon basin characteristics and t_{PR} is the lag time in hours for rainfall duration t_R, calculated as

Eq. 37 $$t_{PR} = t_{pr} + .25(t_R - t_r)$$

In the above equation, t_r is the standard duration of effective rainfall in hours given by

Eq. 38 $$t_r = \frac{t_{pr}}{5.5}$$

and t_{pr} is the lag time from the midpoint of effective rainfall of duration t_r to the peak of a unit hydrograph in hours, computed as

Eq. 39 $$t_{pr} = 0.75 C_t (L L_c)^{0.3}$$

where C_t is a coefficient depending upon basin characteristics, L is the length of stream from the station to the upstream limit of the drainage area in km and L_c is the distance along the main stream from the basin outlet to a point on the stream which is nearest to the centroid of the basin in km.

The coefficients C_t and C_P shall be determined from analysis of some known hydrographs in the region. In the absence of hydrographs, the values of C_t and C_P may be adopted as 1.5 and 0.62, respectively.

11.1.6.3 B.D. Richard's Method

B. D. Richard's method may be used for flood estimation using rainfall and basin characteristics. The method, which has been comprehensively used in Mahakali Irrigation Project in western Nepal, consists of computing the flood discharge Q in m^3/s using the equation

Eq. 40 $$Q = 0.222 AIF$$

where A is basin area in sq. km., I is the rainfall intensity corresponding to the time of concentration T_c and F is an aerial reduction factor given by

Eq. 41 $$F = 1.09352 - 0.6628 \ln(A)$$

The value of I shall be estimated through an iterative procedure in which an initial value of T_c in hours shall be assumed and the following computations performed in sequence:

Eq. 42 $$D = 1.102\frac{L^2}{S}F$$

Eq. 43 $$R_{TC} = 0.22127 R_T T_c^{0.476577}$$

Eq. 44 $$I = \frac{R_{TC}}{T_C}$$

Eq. 45 $$K_R = 0.65 I(T_c + 1)$$

Eq. 46 $$C_{KR} = \frac{0.95632}{K_R^{1.4806}}$$

Eq. 47 $$T_{c3} = DC_{KR}$$

Eq. 48 $$T_{c2} = \left(\frac{T_{c3}}{0.585378}\right)^{\frac{1}{2.17608}}$$

where L is the basin length in km, S is the basin slope, R_T is the 24 hour rainfall for return period T in mm and T_{c2} is the second estimate of the time of concentration. The iterations shall be repeated with $T_c = T_{c2}$ till the difference between the assumed T_c and the resulting second estimate T_{c2} is less than 5%.

11.1.6.4 Fuller's Method

Although developed for basins in the United States of America, Fuller's formula may be used to estimate flood discharges in the ungauged basins of Nepal for comparison purposes. Using this method, the maximum instantaneous flood discharge $Q_{\max}$ in m^3/s shall be estimated as

Eq. 49 $$Q_{\max} = Q_T\left[1 + 2\left(\frac{A}{2.59}\right)^{-0.3}\right]$$

where Q_T is the maximum 24 hour flood with frequency once in T years in m^3/s and A is the basin area in sq. km. Q_T shall be given by

Eq. 50 $$Q_T = Q_{av}(1 + 0.8\log T)$$

in which Q_{av} is the yearly average 24 hour flood over a number of years, in m^3/s, given by

Eq. 51 $$Q_{av} = C_f A^{0.8}$$

where C_f is Fuller's coefficient varying between 0.18 to 1.88. For Nepal, C_f may be taken as the average of these values, i.e. equal to 1.03.

11.1.6.5 Horton's Formula

Horton's formula may be used to compute the flood q_{tr}, in m^3/s/sq. km, equaled or exceeded in a T year return period using the relation

Eq. 52 $$q_{tr} = 71.2\frac{T^{0.25}}{A^{0.5}}$$

where A is the drainage area in sq. km.

11.1.7 Envelope Curves

Envelope curves may be used to obtain quick rough estimations of peak flood values. For this purpose, envelope curves shall be derived by plotting the observed maximum floods for a number of basins in a homogeneous meteorological region against size of basin on log-log paper and enveloping the plotted points by a smooth curve. The peak discharge for a basin of given size shall then be estimated from the curve.

Although such curves are not yet derived for Nepali basins, flood estimates for them may be derived from enveloping curves prepared by Kanwarsain and Karpov for northern and central Indian rivers. Alternatively, the following correlation developed by Baird and McIllwraith based on the maximum-recorded floods throughout the world may be used:

Eq. 53

$$Q_{mp} = 3025 \frac{A}{(278 + A)^{0.78}}$$

In this equation, Q_{mp} is the maximum flood discharge in m^3/s and A is the basin area in sq. km.

11.1.8 Flood Investigation

Flood investigations shall be carried out to collect flood data for the following purposes:

a. Apply and check investigated historical floods data on flood frequency curves.
b. Form supporting evidence for results obtained from other methods for small river basins where little or no information is available.
c. Establish design floods for small mountainous rivers for which the required parameters for flood estimation cannot be easily determined from rainfall data.
d. Determine rough actual floods which occurred in the past and the range of flood flows within which a flood of fixed return period lies.
e. Assess the validity of computational parameters and results derived from other methods.
f. Collect valuable and essential supplementary information for the comprehensive hydrological analysis of the region.

For small run-of-river hydropower projects where no hydro-meteorological data exist, an investigated historical flood, which is in the upper range of discharges, may be directly taken as the design flood.

11.1.8.1 Investigation Parameters

For a flood that has already occurred, the following parameters shall be investigated:

a. Time and location of the flood.
b. Flood peak stage and discharge.
c. Storm conditions and special features of the basin and riverbed.
d. Flood process and flood volume.

11.1.8.2 Types of Investigations

Various types of investigations shall be collected from the field to gather information on the parameters listed in Section 11.1.8.1. The scope and procedure of these investigations shall be as discussed in the following sections.

Time of Occurrence

For river reaches where the density of population is relatively high, the occurrence of large scale floods in the last thirty or forty years may easily be determined by interviewing the senior citizens of the area, who may even reliably recall information or pass on hearsay from several generations back. Apart from this, it may also be helpful to get acquainted with historical documents and facts to get additional hydrological information.

Probable Precipitation Regime

In areas prone to storm floods, data on precipitation shall be collected to estimate the flood process, study regional flood distribution and test the reasonableness of computed results. The precipitation regime investigated shall include the cause of precipitation, the amount of rainfall as well as the precipitation process and trends. A general understanding of the regime may be adequate to supplement the actual flood investigation.

Flood Stage

Information on the flood stage may be obtained from visible flood marks. Such marks may be in the form of floating vegetative matter, such as grass, straw and seeds, left at high water levels, silt lines on riverbanks, traces of erosion on the banks and silt or stain lines on buildings.

Where visible traces to indicate the flood peak stage are not present, tentative flood levels may be obtained from interviews with local residents. However, information from interviews may only be used for reference.

Hydrological Conditions of Basin and River Course

Hydrological conditions of basin and river course shall be collected to quantify four factors, viz. flood stage, surface slope, wetted cross section and riverbed roughness. Its scope shall include the following:

a. Surface shape, flow conditions, bank features, riverbed condition and characteristics, etc.
b. Change of scouring and silting of the river channel cross section.
c. Influence of human activities.

Probable Flood Process

Occasionally, it may be necessary to obtain the design flood process and calculate the total design flood volume. Investigation of the process may involve many difficulties, and the data obtained may only be approximate.

11.1.8.3 Information Processing

The information relating to a particular flood collected during flood investigations shall be crosschecked for consistency, and only reliable data shall be retained. Thereafter, the slope-area method, discussed in Section 7.5.2, shall be used to estimate the flood.

For useful results from the slope-area method, the reach of the river chosen for the flood investigations shall be carefully selected. This reach shall satisfy the following criteria:

a. The quality of high-water marks in the reach shall be good.
b. As far as possible, the reach shall be straight and uniform. Gradually contracting reaches shall be preferred over expanding reaches.
c. The recorded fall in the water surface elevation shall be larger than the velocity head. A fall greater than 0.15 m shall be preferred.
d. The reach shall be as long as possible. Preferably, a length greater than 75 times the mean depth shall be chosen.

The Manning's roughness coefficient n for use in the computation of discharge shall be obtained from standard tables. At times, a relation between n and the stage may be prepared from measured discharges at a neighboring gauging station, and an appropriate value of n may be selected from it, with extrapolation if necessary.

11.2 Estimation of PMF

For a given basin, the PMF shall be estimated by determining the most adverse, plausible meteorological conditions that could be expected to occur in the basin. Consequently, the estimation shall be based on basin-specific data.

Estimation of the PMF shall generally be based on the Probable Maximum Precipitation (PMP) and the unit hydrograph principle. Where the PMP is not available, alternative methods based on envelope curves or empirical relations may be used to estimate the PMF.

11.2.1 Estimation through Probable Maximum Precipitation

Estimation of the PMF based on the Probable Maximum Precipitation (PMP) and the unit hydrograph principle shall be performed using the following procedure:

1. Determination of the duration of design storm:

   The critical period of the design storm shall be taken equal to the time of concentration of the basin.

2. Determination of design storm:

   The PMP shall be estimated through the transposition of selected storms to the basin and their maximization over it. The transposition and maximization shall be achieved through the following steps:

   - Selection and analysis of past major storms on record considered transposable to the basin.
   - Adjustment to the transposable storms for maximum moisture that could occur over the basin.
   - Envelopment of the transposed adjusted storms.

3. Time adjustment of the design rainfall:

   Time adjustment of the design rainfall may be done satisfactorily through a study of the time-distribution pattern of those observed storms in the area for which adequate self-recording gauge data are available by depth-area-duration analysis. The maximum rainfall depths for standard durations of 6, 12, 18, 24, 36 and 48 hours shall be obtained for each of the storms and expressed as percentages of the total storm depth. Enveloping percentages shall then be obtained and applied to adjust the design rainfall based on observational day data.

4. Derivation of design unit hydrograph:

   The design unit hydrograph shall be derived through the following activities:

   - Collection and examination of basic data.
   - Analysis of observed flood hydrograph and sub-division into base flow and direct runoff hydrographs.
   - Analysis of rainfall data related to observed flood hydrographs.
   - Derivation of storm rainfall-runoff relationships from the observed rainfall and runoff records.
   - Derivation of unit hydrographs from observed flood hydrographs.
   - Plotting of all the unit hydrographs and determination of the average unit hydrograph.

5. Determination of critical time sequence of design storm:

   For an exact determination of critical time sequence of the design storm, the increments of rainfall excess shall be arranged opposite to the ordinates of the design unit hydrograph in such a way that the largest increment is opposite the largest hydrograph ordinate, the second largest increment is opposite the second largest ordinate, and so on. This arrangement shall be reversed to obtain the critical sequence.

6. Derivation of PMF hydrograph:

   The critical time sequence of the design storm shall be superimposed on the derived design unit hydrograph to give the direct hydrograph, which when added with the base flow, gives the PMF hydrograph.

11.2.2 Estimation of PMF in Absence of PMP

In areas of sparse data, the PMF may be obtained by drawing envelope curves of the maximum floods recorded in the region under study. In cases where estimates of PMP have not been made, the volumes of rainfall to be expected may also be approximated from envelope curves of the world record rainfalls.

For cases where extreme precipitation data is not available, the following equation suggested by Hersfield may be used to estimate a 24 hour PMP at a point in a region:

Eq. 54 $$PMP_{24} = P_m + KS_n$$

where P_m is mean of 24 hr annual maximum over the period of record, S_n is standard deviation of the 24 hour annual maximum and K is a constant equal to 15. This PMP shall then be superimposed on the design unit hydrograph to find the PMF.

For quick predictions, the PMF may approximately be taken as twice the 10,000 year flood, which is the average of the ratios of PMF to the 10,000 year flood, ranging between 1.34 and 2.94, found from data analysis of many projects throughout the world. For the Karnali basin in western Nepal, this ratio was obtained as 1.91.

11.3 Investigation of GLOF

GLOF investigations shall be performed in basins where its threat is real. The investigations may be conducted by surveying the glacier lake through interpretation of aerial photographs or satellite imageries or through low altitude air surveys using gamma radiation or other sensors.

The main parameters of a GLOF investigation shall include the following:

- Date, time and duration of past GLOF, if any.
- Average depth and area of glacial lake for the calculation of water volume.
- Approximate passing time of the GLOF through the river.

Knowing the total volume and passing time of the potential GLOF, the GLOF discharge shall be estimated approximately.

11.4 Investigation of CLOF

CLOF investigations shall be conducted for basins in the Himalayan regions, particularly if CLOF is known to have occurred in the past in those basins or in surrounding basins. The investigations may be conducted only from past records. During these investigations, the following points shall be considered:

- Date and time of cloudburst.
- Maximum intensity and total time of cloudburst.
- Captured area of cloudburst.
- Passing time of cloudburst flood.

Knowing the total volume and passing time of the CLOF, the CLOF discharge shall be estimated approximately.

11.5 Procedures for Flood Prediction in GRB

Procedures for flood prediction in GRBs shall depend on the data availability in the basin under consideration. For a given class of basin as defined in Table 1, the prediction shall be made using the steps enumerated in the following sections.

11.5.1 Basin G1

If long-term data on hydrology and meteorology are available at the proposed headworks site, flood prediction shall be performed using the following steps:

a. A series of yearly maximum instantaneous flows, i.e. the highest flow in every year, shall be prepared from the daily discharge records. If the flow record is from upstream or downstream of the proposed site, the data may be adjusted simply by the basin area ratio method or any other method.

b. If possible, flood frequency analysis may be undertaken using computer programs.

c. The yearly maximum instantaneous flow values shall be arranged in descending order, and their probability of exceedence p (or return period T) shall be defined using Weibull plotting-position formula

Eq. 55

$$p = \frac{m}{n+1}$$

where m is order number and n is total number of flows.

d. Positions of yearly maximum flows shall be plotted (Q versus p or T) on a probability paper.

e. One or more theoretical probability distributions for extreme events (Gumbel, Pearson type III, log normal or others suitable for flood statistics) shall be used to fit a curve through the plotting positions. The distribution which best fits with the plotting positions shall be adopted for flood prediction through extrapolation. If none of the distributions fit the plotting positions closely, the best fit line of plotting positions shall be used to predict flood values.

f. Flood flow values shall be predicted by extrapolating the selected curve as required for the design return period. Reliability of such prediction shall be excellent.

g. Flood flows may be compared with results from the WECS/DHM method for the proposed site.

h. Flood flows may be calculated by the PCJ method using long-term data on meteorology and compared with the other calculated values.

i. Rainfall-runoff correlation (trend analysis) at the proposed site shall be established from the long-term data on hydrology and meteorology.

j. The maximum historical flood shall be estimated by the slope-area method using information collected from local residents during site visits for flood investigations.

k. GLOF and CLOF investigations shall be carried out to the extent possible.

11.5.2 Basin G2

If long-term data on hydrology is only available at the proposed site, flood prediction shall be made using steps a to g, j and k for Basin G1 listed in Section 11.5.1.

11.5.3 Basin G3

Where short-term data on hydrology at the proposed site and long-term data on hydrology at an HSC are available, the following procedure shall be adopted for flood estimation:

a. A series of yearly maximum instantaneous flows from short-term hydrological data at the proposed site shall be prepared. If the flow record is from upstream or downstream of the proposed site, the data may be adjusted simply by basin area ratio method or any other method.

b. Steps c to f as explained for Basin G1 in Section 11.5.1 shall be carried out. Reliability of this prediction may be very good.
c. Flood flows shall be computed by regional methods (WECS/DHM, MHSP, PCJ, etc.), rational method and empirical formulae (Dickens, Richard, Snyder, etc.).
d. Results obtained from different methods shall be compared, and the flood flow values shall be suitably recommended according to project requirements. Preference shall be given to results from frequency analysis.
e. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.5.4 Basin G4

If short-term data on hydrology and long-term data on meteorology at the proposed site are available, floods shall be predicted using the following procedure:

a. Rainfall-runoff correlation shall be established at the proposed site from the short-term flow and concurrent rainfall data.
b. Flow data shall be extended from the established rainfall-runoff relation.
c. Flood flows shall be estimated by frequency analysis as described for G1 in Section 11.5.1. A suitable extreme value distribution shall be selected by judging the difference between the plotting position and distribution curves, and flood prediction shall be done by extrapolation of the selected theoretical distribution curve. Reliability of this prediction may be very good.
d. Flood flows shall be estimated by regional methods (WECS/DHM, PCJ, MHSP,etc.), rational method and empirical formulae (Dickens, Richard, Snyder, etc.).
e. Results obtained by different methods shall be compared, and the flood flow values shall be suitably recommended according to project requirements. Preference shall be given to results from frequency analysis and regional methods.
f. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.5.5 Basin G5

If short-term data on hydrology at the proposed site and long-term data on meteorology with short-term data on hydrology at an HSC are available, flood predictions at the proposed site shall be achieved as follows:

a. Steps a, b and c for Basin G3 explained in Section 11.5.3 shall be followed.
b. Results obtained by different methods shall be compared and the flood flow values shall be suitably recommended according to project requirements. Preference shall be given to frequency results and regional methods. Reliability of results may be good.
c. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.5.6 Basin G6

If short-term data on hydrology at the proposed site and long-term data on meteorology at an HSC are available, the steps listed below shall be adopted for flood prediction:

a. Positions of short-term maximum instantaneous flood values shall be plotted using Weibull formula, and flood flows shall be estimated by fitting of theoretical distributions.
b. Flood flows shall be estimated by regional methods (WECS/DHM, MHSP, etc.), PCJ method using neighboring rainfall stations outside the basin, rational method and empirical formulae (Dickens, Richard, Snyder, etc.).

c. Results obtained by different methods shall be compared and the flood flow values suitably recommended according to project requirements. Preference shall be given to results obtained by theoretical distribution and regional methods. Reliability of prediction may be good.

d. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.5.7 Basin G7

If only short-term data on hydrology is available at the proposed site, floods shall be estimated through the following steps:

a. Positions of short-term maximum instantaneous flood values shall be plotted using Weibull formula, and flood flows shall be estimated by fitting of theoretical distributions.

b. Flood flows shall be estimated by regional methods (WECS/DHM, MHSP, etc.), rational method and empirical formulae (Dickens, Richard, Fuller, etc.).

c. Results obtained by different methods shall be compared and the flood flow values suitably recommended according to project requirements. Preference shall be given to results obtained by theoretical distribution and regional methods. Reliability of results may be good.

d. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.6 Procedures for Flood Prediction in URB

Procedures for flood prediction in URBs shall depend on the data availability in the basin under consideration. For a given class of basin as defined in Table 2, the prediction shall be made using the steps enumerated in the following sections.

11.6.1 Basin U1

If long-term data on meteorology at the proposed site and long-term data on hydrology at an HSC are available, the following procedure shall be adopted for flood prediction:

a. Collection of flow data shall be immediately started at the proposed site.

b. Floods shall be estimated by regional methods (WECS/DHM, PCJ, MHSP, etc.), the rational method and empirical formulae (Dickens, Richard, Snyder, Fuller, Horton, etc.).

c. Results obtained by different methods shall be compared, and flood flow values shall be suitably recommended according to project requirements. Preference shall be given to the WECS/DHM, PCJ and rational methods. To be on the safer side, the maximum f values obtained from different methods may be adopted. Reliability of such estimates may be good.

d. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.6.2 Basin U2

For basins with long-term data on hydrology at an HSC, flood prediction shall be performed through the following steps:

a. Collection of flow data shall be immediately started at proposed site.

b. Flood flows shall be estimated by regional methods (WECS/DHM, MHSP, etc.), rational method and empirical formulae (Dickens, Richard, Fuller, Snyder, Horton, etc.).

c. For comparison, flood flows may also be estimated by the PCJ method using data from neighboring rainfall stations outside the basin.

d. Results obtained by the different methods shall be compared, and the flood flow values shall be suitably recommended according to project requirements. Preference shall be given to results obtained by the WECS/DHM, rational and Dickens methods. The reliability of such estimates may be poor.

e. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.6.3 Basin U3

If long-term data on meteorology at the proposed site and short-term data on hydrology at an HSC are available, flood prediction shall be performed using the following procedure:

a. Collection of flow data shall be immediately started at the proposed site.
b. Floods shall be estimated by regional methods (WECS/DHM, PCJ, MHSP, etc.), rational method and empirical formulae (Dickens, Richard, Fuller, Snyder, Horton, etc.).
c. Results obtained by different methods shall be compared, and flood flow values shall be suitably recommended according to project requirements. Preference shall be given to results obtained by PCJ and rational methods. Reliability of this estimate may be good.
d. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.6.4 Basin U4

At headworks sites where long-term data on meteorology are only available, the following procedure shall be adopted for flood prediction:

a. Collection of flow data shall be immediately started at the proposed site.
b. Floods shall be estimated by regional methods (WECS/DHM, MHSP, PCJ, etc.), rational method and empirical formulae (Dickens, Richard, Fuller, Snyder, Horton, etc.).
c. Results obtained from different methods shall be compared, and the flood flow values shall be suitably recommended according to project requirements. If they differ considerably, preference shall be given to results obtained by the PCJ and rational methods. If the differences are very small, an average value may be adopted. The reliability of such prediction may be good.
d. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.6.5 Basin U5

If short-term data on meteorology at the proposed site and long-term data on meteorology at an HSC are available, flood prediction shall be achieved as follows:

a. Collection of flow data shall be immediately started at the proposed site.
b. Flood flows shall be estimated by regional methods (WECS/DHM, MHSP, etc.), rational method and empirical formulae (Dickens, Richard, Fuller, Snyder, Horton, etc.).
c. Flood flows may be estimated by the PCJ method at the HSC and transferred to the proposed site by the basin area ratio method.
d. Results obtained by different methods shall be compared, and the flood flow values shall be suitably recommended according to project requirements. If they differ considerably, preference shall be given to results obtained by the regional methods and the rational method. If the difference is very small, an average of the values from the different methods may be adopted. The reliability of such prediction may be satisfactory.
e. Steps j and k for Basin G1 listed in Section 11.5.1 shall be carried out.

11.6.6 Basin U6

If short-term data on meteorology only are available at an HSC, floods shall be predicted as follows:

a. Collection of flow data shall be immediately started at the proposed site.

b. Flood flows shall be estimated by regional methods (WECS/DHM, MHSP, etc.) and empirical formulae (Dickens, Richard, Fuller, Snyder, Horton and others).
c. Flood flows shall be estimated at the HSC by the rational and PCJ methods and transferred to he proposed site by the basin area ratio method.
d. The maximum historical flood shall be estimated by the slope-area method using information collected from local residents during site visits for flood investigations.
e. Results from different methods shall be compared, and the flood flow values shall be suitably recommended according to project requirements. If they differ considerably, preference shall be given to results obtained by regional methods. If the difference is very small, an average value may be adopted. The reliability of such prediction may be poor.
f. GLOF and CLOF investigations shall be carried out to the extent possible.

11.6.7 Basin U7

For basins where no data are available, the following procedure shall be adopted for flood prediction:

a. Collection of flow data shall be immediately started at the proposed site.
b. Flood flows shall be estimated by regional methods (WECS/DHM, MHSP, etc.) and by empirical formulae (Dickens, Richard, Fuller, Snyder, Horton, etc.), by the rational method using suitable assumptions for the rainfall intensity and runoff coefficient and with the help of envelope curves developed for Northern Indian basins referred to in Section 11.1.7.
c. The maximum historical flood shall be estimated by the slope-area method using information collected from local residents during site visits for flood investigations.
d. At least one direct measurement of flood discharge shall be conducted at the proposed site.
e. Results obtained by different methods shall be compared, and the flood flow values shall be suitably recommended according to project requirements. If they differ considerably, preference shall be given to results obtained by regional methods, rational method and empirical methods. If the difference is very small, an average value may be adopted. The reliability of such prediction may be very poor.
f. GLOF and CLOF investigations shall be carried out to the extent possible.

11.7 Important Considerations in Flood Prediction

While predicting flood using the methods discussed in the preceding sections, the following issues shall be borne in mind:

- Only instantaneous maximum values shall be used for frequency analysis.
- Flow data transposition from an HSC shall be avoided.
- If adequate data is not available for frequency analysis, regional and/or empirical methods shall be used.
- Where practicable and desirable, PMF based on the PMP shall be determined.

12. LONG-TERM HYDROLOGY

If long-term data on monthly flows are available, average of the monthly flows of annual series for any month shall give the long-term stream flow for that month, the reliability of this estimate being excellent. In the absence of monthly flow data regional methods, data transposition from an HSC and empirical relations shall be used, but the reliability of such estimates shall decrease up to very poor as in the case of no data at all.

12.1 Long-term Stream Flow Analysis

For the assessment of long-term hydrology, the following graphs shall be derived based on monthly flows of annual series for all twelve months:

a. Flow Duration Curve.

b. Mean Monthly Hydrograph.

The flow duration curve shall be prepared by plotting values of stream flow (daily, weekly, or monthly) in order of magnitude as ordinates and percent of time as abscissas. It may be plotted on the calendar year basis or the total period basis.

Mean monthly hydrograph shall be plotted for average, dry and wet years as required. It shall be prepared the calendar months on the abscissa and the long-term average flow for the particular month on the ordinate.

12.2 Regional Methods for Estimation of Monthly Flows

In the absence of monthly flow data, the following regional methods developed for Nepal may be used:

a. MIP Method (Sir M. MacDonald & Partners, 1990).

b. WECS/DHM Method (WECS/DHM, 1990).

12.3 Procedures for Prediction of Mean Monthly Flows in GRB

Procedures for long-term flow prediction in GRBs shall depend on the data availability in the basin under consideration. For a given class of basin as defined in Table 1, the prediction shall be made using the steps enumerated in the following sections.

12.3.1 Basin G1

a. Yearly series of mean monthly flows shall be prepared for 12 months by taking mean of the daily flows of particular month. If the flow record is from upstream or downstream of the proposed site, the data may be adjusted simply by basin area ratio method.

b. Long-term mean monthly flow for all 12 months can be calculated by taking mean of the annual series for particular month. These mean monthly flows shall be used for making mean monthly hydrograph and flow duration curve. Reliability of this prediction will be excellent.

c. Rainfall–runoff correlation shall be established by concurrent rainfall and runoff data available at the proposed site.

d. WECS/DHM and MIP regional methods may be used to determine long term mean monthly flows for 12 months to compare the values.

12.3.2 Basin G2

a. Yearly series of mean monthly flows shall be prepared for 12 months by taking mean of the daily flows of particular month. If the flow record is from upstream or downstream of the proposed site, the data may be adjusted simply by basin area ratio method.

b. Long-term mean monthly flow for all 12 months can be calculated by taking mean of the annual series for particular month. These mean monthly flows shall be used for making mean monthly hydrograph and flow duration curve. Reliability of this prediction will be excellent.

c. WECS/DHM and MIP regional methods may be used to determine long term mean monthly flows for 12 months to compare the values.

12.3.3 Basin G3

a. Short-term stream flow data at proposed site shall be extended from long-term stream flow data at HSC using methods of data extension.

b. Steps a and b of G1 shall be followed to calculate mean monthly flows of the annual series from the extended data. Reliability of this estimate will be very good if data has been extended carefully.

c. Mean monthly flows shall be estimated by WECS/DHM and MIP regional methods.

d. Appropriate values of mean monthly flows shall be selected. A careful analysis is needed to select the values. Importance shall be given to extended flow data from HSC. Reliability of prediction may be very good.

12.3.4 Basin G4

a. Rainfall-runoff correlation at proposed site shall be established from short-term flow and concurrent rainfall data of proposed site.

b. Flow data shall be extended from established rainfall-runoff relation. Reliability of such extension depends upon the methods of data extension.

c. Steps a and b of G1 shall be followed to calculate mean monthly flows of the annual series from the extended data.

d. Mean monthly flows shall be calculated by WECS/DHM and MIP regional methods.

e. Appropriate values of mean monthly flows shall be selected. Importance shall be given to extended flow data from rainfall. Reliability of prediction may be very good.

12.3.5 Basin G5

a. Relationship shall be developed between short-term flow data at proposed site and HSC.

b. Rainfall-runoff correlation shall be established at HSC from short-term flow and concurrent rainfall data available at HSC.

c. Flow data at HSC shall be extended from established rainfall-runoff relation at HSC.

d. Flow data at proposed site shall be extended from long-term flow data of HSC obtained in step c using the relationship between short-term flow data at proposed site and HSC established in step 1.

e. Steps a and b of G1 shall be followed to calculate mean monthly flows of the annual series from the extended data.

f. Estimate of mean monthly flows shall be made by WECS/DHM and MIP regional methods.

g. Appropriate values of mean monthly flows shall be selected. If the differences are considerable, selection of extended data from HSC will be appreciated. Reliability of this estimate can be considered as good.

12.3.6 Basin G6

a. If short-term flow data at proposed site is concurrent to long-term rainfall data at HSC then a relationship between flow of proposed site and rainfall of HSC should be established and flow data at proposed site should be extended from rainfall at HSC, using the established relationship.

b. If data are not concurrent, then the long-term stream flow at HSC shall be first estimated from long-term rainfall of HSC with the help of empirical formulae or other methods, then the estimated long-term stream flow of HSC shall be transposed to the proposed site and may be combined with the short-term flow data of proposed site.

c. Steps a and b of G1 shall be followed to calculate mean monthly flows of the annual series from the established data.
d. Estimate of mean monthly flows shall be made by WECS/DHM and MIP methods.
e. Appropriate values of mean monthly flows shall be selected. If the values differ considerably, then selection of generated data is more appreciable. Reliability of such estimates may be good.

12.3.7 Basin G7

a. Mean monthly flows shall be calculated from available flow data at proposed site.
b. Estimate of mean monthly flows shall be made by WECS/DHM and MIP regional methods.
c. Average values of mean monthly flows by different methods may give satisfactory results. Reliability of such estimates may be acceptable (satisfactory).

12.4 Procedures for Prediction of Mean Monthly Flows in URB

Procedures for long-term flow prediction in URBs shall depend on the data availability in the basin under consideration. For a given class of basin as defined in Table 2, the prediction shall be made using the steps enumerated in the following sections.

12.4.1 Basin U1

a. Collection of flow data shall be started immediately at proposed site.
b. Flow data at proposed site shall be established from long-term stream flow data at HSC using methods of data transposition. Mean monthly flows shall be calculated from established flow data at proposed site.
c. Mean monthly flows shall be estimated from rainfall data of proposed site by different empirical methods or other suitable methods.
d. Mean monthly flows shall be estimated by WECS/DHM and MIP methods.
e. Appropriate values of mean monthly flows shall be selected. Priority shall be given to the result obtained through HSC. Reliability of such estimates will be very good if the HSC has been selected properly.

12.4.2 Basin U2

a. Collection of rainfall and flow data shall be started immediately at proposed site.
b. Flow data at proposed site shall be established from long-term stream flow data at HSC using methods of data transposition. Mean monthly flows shall be calculated from established flow data at proposed site.
c. Mean monthly flows shall be estimated by WECS/DHM and MIP methods.
d. Appropriate values of mean monthly flows shall be selected. Priority shall be given to the result obtained through HSC. Reliability of such estimates will be very good if the HSC has been selected properly.

12.4.3 Basin U3

a. Collection of flow data shall be started immediately at proposed site.
b. If long-term data on meteorology at proposed site is concurrent to short-term data on hydrology at HSC then a relationship shall be established between rainfall of proposed site and runoff of HSC, and flow data at HSC shall be extended from rainfall of proposed site, using the established relationship. Now flow data of HSC shall be transferred to the proposed site.
c. If data are not concurrent, then the short-term data on hydrology of HSC shall be transferred to the proposed site. Long-term data on hydrology at proposed site shall be

estimated from long-term data on meteorology of proposed site with the help of empirical formulae. Combination of both flow data may be used as long series.

d. Mean monthly flows shall be estimated by WECS/DHM and MIP methods.
e. Appropriate values of mean monthly flows shall be selected. Priority shall be given to transposed data from HSC. Reliability of such estimates may be good.

12.4.4 Basin U4

a. Collection of flow data shall be started immediately at proposed site.
b. Mean monthly flows at proposed site shall be estimated from long-term data on meteorology of proposed site with the help of empirical formulae such as Khosla and UPIRI.
c. Mean monthly flows shall be estimated by WECS/DHM and MIP methods.
d. Appropriate values of mean monthly flows shall be selected. Average of values obtained by different methods may be adopted for better results. Reliability of this estimate will be satisfactory.

12.4.5 Basin U5

a. Collection of flow data shall be started immediately at proposed site.
b. Short-term data on meteorology of proposed site and long-term data on meteorology of HSC shall be correlated and rainfall data at proposed site shall be generated from long-term data on meteorology of HSC.
c. Mean monthly flows at proposed site shall be estimated from generated long-term data on meteorology at proposed site with the help of empirical formulae such as Khosla and UPIRI.
d. Mean monthly flows shall be estimated by WECS/DHM and MIP methods.
e. Appropriate values of mean monthly flows shall be selected. Average of flow values obtained by different methods may give better results. Reliability of this estimate will be satisfactory provided HSC has been selected carefully.

12.4.6 Basin U6

a. Collection of rainfall and flow data shall be started immediately at proposed site.
b. Short-term data on meteorology from HSC shall be transferred to proposed site.
c. Mean monthly flows at proposed site shall be estimated from transferred Short-term data on meteorology at proposed site with the help of empirical formulae such as Khosla and UPIRI.
d. Mean monthly flows shall be estimated by WECS/DHM and MIP methods.
e. Appropriate values of mean monthly flows shall be selected. If results obtained from transferred rainfall data from HSC are differing highly in comparison with WECS/DHM and MIP methods, then average of WECS/DHM and MIP methods are recommended for better results. Reliability of this estimate will be poor.

12.4.7 Basin U7

a. Collection of rainfall and flow data shall be started immediately at proposed site.
b. Mean monthly flows shall be estimated by WECS/DHM and MIP methods.
c. Average of WECS/DHM and MIP flow values may be adopted but reliability of this estimate will be very poor.

12.5 Important Considerations in Prediction of Monthly Flows

While predicting monthly flows, the following considerations shall be borne in mind:

a. In general, averaging of results from two or more methods may not be done, but the hydrologist may chose to do so with strong justification.

b. Wherever reliable flow and/or precipitation data are available for a nearby site or an HSC, the WECS/DHM method or MIP method may be used for comparison only.

c. Wherever possible, available precipitation data of adjacent and/or neighboring basins may be used in the analysis when data for the same catchment is not available.

d. Climatic atlases, which give isohyets of precipitation, may be used before attempting regional methods.

13. LOW FLOW HYDROLOGY

Low flow estimation shall basically depend upon data availability. If long-term minimum instantaneous flow data are available in a GRB, statistical frequency analysis shall be used to estimate the low flow of required probability, and the reliability of this estimate may be excellent. In the worst condition of data availability, i.e. when there is no data at all, the prediction may be made using regional methods and empirical formulae, but the reliability of prediction in this case will be very poor.

13.1 Methods of Low Flow Estimation

Low flow estimation may be performed using the following methods:

a. Frequency Analysis Methods for Low Flows and Treatment of Zeros

b. Regional Estimates of Low Flow Statistics

c. Regional Regression Procedures

d. Basin Area Ratio Method

e. Regional Statistics Methods

f. Base Flow Correlation Procedures

g. Recession Curve Analysis

13.2 Low-Flow Investigation

Low-flow investigation shall be performed to obtain low flow data in ungauged regions. It may also be used to restore the recorded low flow to the original value and to obtain practical information concerning the influence of human actions. The contents of a low-flow investigation shall include:

a. Drought conditions in history and extent of disaster

b. Contributing sources to the low water flow

c. Discharge and stage in the year with catastrophic or relatively serious droughts

d. Times and duration when the river dried up and flow ceased

e. Other human activities such as irrigation, water supply, construction of dams etc.

13.3 Regional Methods for Low flow Estimation

Regional analysis of low flows for Nepal may be conducted using the WECS/DHM method (WECS/DHM, 1990).

13.4 Empirical Formulae for Low Flows

For Nepali river basins, Goroshkov's formula may be used to predict low flows. Using this method, the minimum monthly flow discharge of 80% reliability, $Q_{\min(80\%)}$ in m^3/s, shall be computed as

Eq. 56 $$Q_{\min(80\%)} = a(A + f)^n$$

where is A is the drainage area in sq. km., f is a part of the catchment area feeding the river with additional flow due to the presence of springs (taken as 5 to 10% of A), a is a drainage area coefficient (= 0.0014 for winter season) and n is a rainfall coefficient (= 1.27 for winter season). The minimum discharge $Q_{\min}$ for any probability p shall then be computed as

Eq. 57
$$Q_{\min(p\%)} = \lambda Q_{\min(80\%)}$$

where λ is a transition coefficient. Values of λ shall be chosen from Table 8:

Table 8: Values of transition coefficient for different probabilities

p (%)	75	80	85	90	95	97
λ	1.04	1.00	0.94	0.87	0.80	0.75

13.5 Procedures for Prediction of Low Flows in GRB

Procedures for low flow prediction in GRBs shall depend on the data availability in the basin under consideration. For a given class of basin as defined in Table 2, the prediction shall be made using the steps enumerated in the following sections.

13.5.1 Basin G1

a. A series of yearly instantaneous minimum flows (1day, 7days, 30days or monthly) of required period shall be prepared. If the flow record is from upstream or downstream of the proposed site, the data may be transferred simply by Basin Area Ratio method or any other method.
b. Computer programs may be used to undertake low flow frequency analysis on each of these four series, by fitting a type III extreme value distribution.
c. Flow data shall be arranged in ascending order and their probability of occurrence, p shall be defined using Eq. 55.
d. Plotting positions of yearly minimum flows (Q versus p) shall be plotted in a probability paper.
e. Extreme value type III probability distribution for extreme events shall be fitted and compared with observed data.
f. A selection shall be made between two curves: plotting position curve and probability distribution curve by judging the difference between two curves. If they are close then extrapolation shall be done from probability distribution curve otherwise best fit line of plotting positions shall be used.
g. Low flow values shall be estimated by extrapolating the selected curve as per requirement of design return period. Reliability of low flow estimate may be excellent in this case.
h. Estimate of low flows may be compared with WECS/DHM regional method and Goroshkov's low flow formula.

13.5.2 Basin G2

a. Similar procedure shall be followed as explained for G1.

13.5.3 Basin G3

a. Short-term data on hydrology at proposed site shall be extended from long-term data on hydrology at HSC using methods of data extension.
b. Low flows shall be estimated by frequency analysis as described for G1.
c. Low flows shall be estimated by WECS/DHM regional method and by empirical formulae such as Goroshkov's formula.

d. Low-flow investigations shall be carried out at the site to the maximum extent possible.
e. Low flow results shall be compared and suitably recommended according to project requirements. Priority shall be given to result obtained by extended flow data. Reliability of such estimate is very good provided low flow values obtained by different methods are similar to each other.

13.5.4 Basin G4

a. Positions of short-term data on hydrology shall be plotted on a probability paper and low flows shall be predicted by fitting the theoretical probability distribution. Selection shall be made between two curves: Plotting position curve and probability distribution curve, by judging the difference between two curves. If they are close then extrapolation shall be done from probability distribution curve otherwise best fit line of plotting positions shall be used.
b. Low flows shall be estimated by WECS/DHM regional method and by empirical formulae such as Goroshkov's formula.
c. Low-flow investigations shall be carried out at the site to the maximum extent possible.
d. Comparison shall be made of low flow values estimated by different methods.
e. Low flows shall be recommended with proper justification. The safe side is to select the lowest values. Reliability may be good, provided probability distribution fits the short-term low flow data.

13.5.5 Basin G5

a. Basin Area Ratio of proposed site to HSC shall be determined.
b. Short-term data on hydrology of HSC shall be transferred to proposed site based on basin area ratio.
c. All flow values of proposed site and flow values transferred from HSC shall be plotted on a probability paper, and the low flows shall be estimated by fitting the theoretical probability distribution.
d. Low flows shall be estimated by WECS/DHM regional method and by Goroshkov's formula.
e. Low-flow investigations shall be carried out at the site as more as possible.
f. Comparison shall be made of low flow values estimated by different methods.
g. Low flows shall be recommended with proper justification. The safe side is to select the lowest values. Reliability of such estimate may be good.

13.5.6 Basin G6

a. Steps of G4 shall be followed.

13.5.7 Basin G7

a. Steps of G4 shall be followed.

13.6 Procedures for Prediction of Low Flows in URB

Procedures for low flow prediction in uRBs shall depend on the data availability in the basin under consideration. For a given class of basin as defined in Table 2, the prediction shall be made using the steps enumerated in the following sections.

13.6.1 Basin U1

a. Collection of flow data shall be started immediately at proposed site.
b. Flow data from HSC shall be transposed to proposed site by Basin Area Ratio method or any other method.

c. Low flow values shall be estimated by plotting positions and fitting of theoretical distribution as explained for G1.
d. Low flows shall be estimated by WECS/DHM regional method and by Goroshkov's formula.
e. Low-flow investigations shall be carried out at the site as more as possible.
f. Comparison shall be made of low flow values estimated by different methods.
g. Low flows shall be recommended with proper justification. The safe side is to select the values obtained from frequency analysis. Reliability of such estimate may be satisfactory.

13.6.2 Basin U2

a. Steps of U1 shall be followed.

13.6.3 Basin U3

a. Collection of flow data shall be started immediately at proposed site.
b. Short-term flow data from HSC shall be transposed to proposed site by Basin Area ratio method or any other method.
c. Low flow values shall be estimated by plotting positions and fitting of theoretical distribution as explained for G1.
d. Low flows shall be estimated by WECS/DHM regional method and Goroshkov's formula.
e. Low-flow investigations shall be carried out at the site as far as possible.
f. Comparison shall be made of low flow values estimated by different methods.
g. Low flows shall be recommended with proper justification. The safe side is to select the lowest values. Reliability of such estimate may be poor.

13.6.4 Basin U4

a. Collection of flow data shall be started immediately at proposed site.
b. Low flows shall be estimated by WECS/DHM regional method and Goroshkov's formula or other formulae.
c. Low-flow investigations shall be carried out at the site as more as possible.
d. Comparison shall be made of low flow values estimated by different methods.
e. Low flows shall be recommended with proper justification. The safe side is to select the lowest values. Average of low flow values obtained by different methods may be taken, if the differences are minimal. Reliability of such estimate will be very poor.

13.6.5 Basin U5

a. Steps of U4 shall be followed.

13.6.6 Basin U6

a. Steps of U4 shall be followed.

13.6.7 Basin U7

a. Steps of U4 shall be followed.

13.7 Important Considerations in Low Flow Prediction

While predicting low flows, the following considerations shall be borne in mind:

a. Extreme caution to be exercised in selecting the HSC whose data shall be used for transposition to the proposed site.

b. If adequate data for frequency analysis is not available, and transposition of data from a HSC is not possible, regional methods, empirical techniques and recession curve analysis shall be used.

14. RATING CURVES AT HEADWORKS SITE

The stage-discharge relation, or the rating curve, at the headworks shall be developed by plotting measured discharges as the abscissa and the corresponding stages as the ordinate. The discharges used for plotting the curves shall be obtained from current meter or other direct flow measurement methods discussed in Section 7.5.1.

Since the flood period in Nepal is well defined (monsoon months of June to September), the bed rugosity and water surface slope conditions of Nepali rivers are different in the monsoon and non-monsoon months. As such, it shall be desirable to develop separate stage discharge rating curves for these periods.

14.1 Extension of Rating Curve

Due to gauging within a limited range of gauge heights, rating curves may not cover the full range of gauge heights. In such cases, the curve may have to be extended downwards for the low stages and upward for the flood stages. Such extrapolations beyond the range of actual observations shall be made with caution to prevent the introduction of serious errors due to the possibility of a change of control in the extended range.

Extension of rating curves may be performed using any one of the methods discussed in the following sections. To guard against possible errors due to extrapolation, the extension shall always be checked with results from more than one of these methods.

14.1.1 Steven's Method

Using Steven's method, the discharge Q through a river cross section shall be assumed to be a function of $AD^{0.5}$, where A is the area of cross section of the river and D is the mean depth of flow. Considering that A and D are both functions of the gauge height, a curve between Q versus $AD^{0.5}$ shall be plotted for all simultaneous observations of discharges and stages. By extending this curve upwards, discharges for higher stages shall be estimated.

14.1.2 Logarithmic Method

The logarithmic method shall be used if the cross-section of a stream at the gauge site is, or approximates to, a uniform section that can be roughly fit to a segment of a circle, parabola, rectangle or trapezoid. In this case, the discharge shall be expressed as

Eq. 58

$$Q = C\left(G - G_0\right)^n$$

where Q is the discharge, G is the gauge height, G_0 the is the gauge height corresponding to zero discharge and C and n are constants for the station. Writing this equation in logarithmic form as $\log Q = \log C + n \log (G - G_0)$, a straight line shall be fitted through a plot of all simultaneous observations of discharges. This line may then be used for estimating higher or lower discharges.

14.1.3 Manning's Formula

The use of Manning's formula to extend rating curves may be made under the assumption that the product of $S^{1/2}/n$, where S is the slope and n the rugosity coefficient, attains a stable value for a maximum observed discharge and remains constant beyond the range of observed discharges. With this assumption, a procedure similar to Steven's method (Section 14.1.1) shall be adopted to extend the rating curve.

14.2 Procedure for Constructing Rating Curve

For the construction of rating curves at the headworks sites, following procedures shall be adopted:

a. Points at which the stage-discharge curves are required shall be detailed. Normally, they may be required at the foot of all hydraulic structures and also at control points for flood control purposes.
b. The approach for constructing the curve – whether hydrologic or hydraulic – shall be decided according to data availability.
c. For the hydrologic approach (fitting a curve to actual observed stages and discharges), the reliability of data and the fitting techniques, including the choice of zero of the gauge (elevation corresponding to zero discharge), shall be discussed. The stability of the curve with reference to historical data shall be discussed. Possibilities of shifting controls shall also be discussed.
d. For the hydraulic approach, reaches of interest shall be defined. Assumptions regarding the coefficient of rugosity, or details of studies for determining it, shall be discussed. Non-uniform steady flow computations shall be carried out from downstream to the required point for a set of discharges.
e. Whether the initial water level at the downstream end are assumed or are based on known gauge-discharge ratings derived from actual discharge measurements shall be discussed.
f. For important hydraulic structures, the likely upper and lower limits of the stage-discharge rating shall be computed either as statistical confidence limits or by making different assumptions on rugosity, etc. within acceptable range.
g. In both hydrologic and hydraulic approaches, additional allowance to cater for the loop rating effect during the passage of a flood wave may also be added when using the curve for determining design flood levels.

15. WATER SURFACE PROFILE AT HEADWORKS SITE

For preparing the water surface profile, three cross-sectional surveys at the headworks site shall be carried out covering the highest flood marks. Preferably, these surveys shall be made at the locations surveyed during previous studies so that changes in the cross sections, if any, can be observed and the magnitude of flood peaks can be checked with the previous ones.

Rating curves shall be developed for the headworks site. From these curves, the water surface profiles shall be estimated for the required discharges.

16. DOCUMENTATION

Upon completion, hydrological investigations shall be documented in adequate detail to allow the headworks designer to comprehend the bases of the investigations and use its results with sufficient confidence. The documentation shall include, but not be limited to, the following aspects of the hydrological investigations and studies:

- Basin description and characteristics, including its location, size, elevation and shape, steepness of its terrain, slope and length of the main water course, vegetation cover, permeability of soil, other basin characteristics, snow area, basin area below 5,000 m, etc.
- Availability of data on stream flow, precipitation, snow, rainfall pattern, GLOF and CLOF, temperature, wind, glacial flow, including the length of record and data quality.
- Field investigations, including discharge measurement, trash marks, river cross-sections at the headworks site, high flood levels, river slope, estimate of Manning's *n*, downstream water rights, etc.

- Identification and verification of HSC, including its location, size, shape, elevation and other basin characteristics.
- Availability of data in HSC on stream flow, precipitation, snow, rainfall pattern, etc., including the length of record and data quality.
- Review of past hydro-meteorological studies within the project area.
- Establishment and selection of a reliable hydrological and meteorological database for the estimation of design flows, including the average daily and monthly flows for the FDC and hydrographs.
- Low flow estimation, including selection of methods according to data availability, simulation or computation by different methods, comparison of results obtained from different methods and recommendation of design low flows for different return periods with justification.
- Long-term mean flow estimation, including selection of methods according to data availability, simulation or computation by different methods, comparison of results obtained from different methods, recommendation of long-term mean flows with justification.
- High flow estimation, including selection of methods according to data availability, simulation or computation by different methods, comparison of results obtained from different methods, recommendation of high flows for different return periods with justification, estimation of PMF and construction diversion flood.
- Flow duration curve, including calculation of probability of flows and plotting of FDC.
- Average annual hydrograph.
- Stage-discharge relation (rating curve) and water surface profiles at the headworks.

PART 1C – SEDIMENTOLOGICAL INVESTIGATION

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Part

1C

Sedimentological Investigations

1. PURPOSE

Part 1C of the *Design Guidelines for Headworks for Hydropower Projects* establishes procedural guidelines for sedimentological investigations performed in support of the design of headworks of run-of-river hydropower projects. The guidelines are intended to ensure adoption of uniform procedures in such investigations.

2. SCOPE

The guidelines focus on the desirable quality and quantity of sedimentological investigations required for design of headworks of run-of-river hydropower projects. They discuss the planning considerations and principal methods of investigation for acquiring the data and information on the sedimentological features and design parameters of the settling basin. They also define standard procedures for the analysis, processing and presentation of these investigations.

3. TERMINOLOGY

Terms used in these guidelines are defined below:

Bed load	All particles that mainly move close to the riverbed by sliding, rolling and jumping.
Sediment rating curve	Correlation between the sediment concentration and the water discharge.
Suspended load	All particles that mainly are carried by the water flow in suspension and travel with more or less the same velocity as the water flow.
Sediment yield	Amount of sediments transported by a river or a stream through a given cross section divided by the catchment area on an annual basis.

Sediment concentration Amount of sediments in the water flow.

4. OBJECTIVE OF INVESTIGATIONS

Sedimentological investigations shall be essentially conducted to establish the quantity and quality of suspended sediments and bed load that the river is likely to carry at the proposed site of headworks of run-of-river projects.

5. SCOPE OF INVESTIGATIONS

To attain the objectives listed in Section 4, the activities listed below shall normally be performed as part of sedimentological investigations:

a. Sediment sampling.
b. Laboratory analysis of samples.
c. Estimation of sediment loads.
d. Prediction of sediment yield.

6. INVESTIGATION PLANNING

Before their commencement, sedimentological investigations shall be adequately planned to define their scope and extent. The planning shall be accomplished primarily through desk studies and site visits conducted jointly by the headworks design team and the investigation team.

6.1 Planning Considerations

Sedimentological investigations shall be planned considering the data requirements for hydraulic design, seasonal variability in sediment load and the project development schedule.

Data Requirements for Headworks Design

Sedimentological investigations shall be geared towards generating appropriate sediment data needed specifically for the design of headworks of run-of-river schemes. These data shall include:

- Concentration of suspended sediments in the water flow.
- Particle size distribution of suspended sediments.
- Bed load transport rates.
- Particle size distribution of bed load.
- Particle size distribution of the riverbed material / armored layer.
- Mineralogical and petrographical composition of the sediment load, i.e. the contents of hard minerals like quartz, garnet and feldspar.
- Content of organic matter in suspended load.
- Density of deposited sediments.

Seasonal Variation

Sedimentological investigations shall be drawn up considering the seasonal variations in sediment transport rates in Nepali rivers resulting from rainfall pattern, snow melting and weathering. As the bulk of the total sediment transport occurs during the monsoon months, a larger number of measurements shall be planned during these months.

Schedule

To ensure that the headworks design is based on adequate and reliable sediment data, sediment sampling shall be initiated at the headworks site at an early stage of headworks development, preferably the pre-feasibility or feasibility stage of the project. Collection of data shall be planned to be continued during the design phase as well.

6.2 Planning Data and Information

Sedimentological investigations shall be planned based on the available information on the drainage area characteristics and the sediment characteristics of the river basin. Such data and information shall be obtained from the sources indicated in the following sections.

Drainage Area Characteristics

Information on the physiographic characteristics of the drainage area needed for sediment load estimation shall be derived from 1:25,000 and 1:50,000 topographic maps published by the Survey Department, GoN. Similarly, information on geological formations and lithology of the basin shall be obtained from regional geological maps published by the Department of Mines and Geology, GoN, and engineering geological maps prepared by the Department of Irrigation, GoN, and other agencies involved with hydropower development in Nepal.

Sediment Data

Limited data on sediments in some river basins may be obtained from the Department of Hydrology and Meteorology (DHM), GoN. Some sediment-related data of Nepali rivers, such as recorded concentrations, estimated sediment load, developed rating equations and predicted mean monthly loads, may also be obtained from different literature and reports on hydropower projects studied in the river basin being considered. Such reports are available for projects studied up to different levels by the Nepal Electricity Authority, DoED, Water and Energy Commission, the erstwhile Electricity Department and private hydropower developers. Sediment data may also be available at hydropower projects under operation in the river basin. However, any useful data from such sources shall be used after proper scrutiny.

7. SEDIMENT SAMPLING

Sediment sampling shall consist of sampling of bed load and suspended sediments.

7.1 Bed Load Sampling

Bed load sampling shall be conducted by catching the moving bed load in a trap or a basket using different equipment. For reliable measurement, the sampling shall generally involve 20 to 100 individual measurements, from 4 to 10 points across the river width, repeated over a range of river discharges. Under specifically required for a particular project, such measurements, owing to the difficulty in conducting them, may be performed only for major projects.

7.1.1 Sampling Methods

Sampling of bed material shall be conducted using one or more of the following methods:

a. Grab sampling.
b. Surface sampling.
c. Pit samples.

7.1.1.1 Grab Sampling

Grab samples of the river bed, consisting of gravel and sand, shall be taken by wading from a boat or a cable way. The samples shall be obtained using scoop-type samplers or digging samplers. If the river consists of coarse material, a clamshell bucket sampler shall be used.

7.1.1.2 Surface Sampling

Surface sampling shall be performed to collect data for assessing when the actual surface material will move. For this, surface samples shall normally be taken from exposed shoals. Sampling may be performed by defining a grid and picking up particles from each grid intersection. For larger areas, sampling may be performed along a line, or several lines, chosen by eye to be typical of that reach of the river. Samples shall be taken by walking along the lines and picking up the stone under the right foot at every other stride.

In a gravel river where coarser materials can form a layer and thus armor the bulk of the bed material, surface samples may not be representative of the material in depth required for assessing the movement of actual surface material. In these cases, a bulk sample shall be taken by digging a hole in order to assess sediment transport after the break up of the armor layer.

7.1.1.3 Pit Sampling

Pit sampling shall be conducted by trapping of the material in a pit or trench excavated across a river. Owing to the expense and difficulty in excavating the pit, this method may generally be used in small rivers only.

7.1.2 Sampling Frequency

The sampling frequency of bed material shall depend on the sampling method adopted. In case of pit sampling, the sampling frequency may be once per year. For the other methods, sampling may be performed two or three times per year.

7.2 Suspended Load Sampling

For estimation of the suspended sediment load, samples shall be taken at the headworks site in a regular pattern across the river and at a range of depths. The sampling shall then be repeated for different discharges so that a reliable and representative time series of the sediment concentration and particle size distribution of the suspended load is produced. The sampling shall also be repeated to account for possible extreme situations, such as floods, that could result in extreme sediment concentrations even for short durations.

7.2.1 Sampling Methods

Depending on the size of the river, suspended sediment samples may be obtained by wading in a boat or from a cableway. The sampling may be done using the methods discussed in the following sections.

7.2.1.1 Instantaneous Sampling

Instantaneous sampling shall be performed using a horizontal tube open at both ends. The tube shall be lowered to the required depth, and its ends shall then be closed to collect the sample.

7.2.1.2 Time-Integrated Point Sampling

In this technique, the sample shall be collected by filling a tube over a short but significant time interval so that fluctuations of sediment concentration are also represented. Since the sample is representative of only one point in the cross section, samples shall be collected at selected depths at stream verticals representing areas of equal water discharge in the cross section.

7.2.1.3 Depth-Integrated Sampling

In this method, the sampler shall be lowered to the streambed and then raised to the surface at a constant rate so that a discharge-weighted mean concentration for the vertical location is obtained. The rate of rising of the device shall be chosen by trial and error so that the sampler is not completely filled on its return to the surface. The verticals across the river shall be selected as for the time-integrated point sampling method. The rate of rise on a vertical must be constant but need not be the same as on other verticals.

7.2.1.4 Single Stage Sampling

This type of sampling shall be performed where a sample is required at high flows. In this method, a simple unmanned sampler, e.g. a bottle, shall be mounted at a predetermined location and level above the normal water surface so that the sampler can take the sample slowly on the rising stage.

7.2.1.5 Pumping Sampling

In this method, an automatic sampler shall be programmed to abstract samples at pre-set time intervals. Alternatively, it may be activated when the water level reaches a certain stage.

7.2.2 Sampling Procedure

While sampling using any of the methods discussed in Section 7.2.1, the required sampling instrument shall be positioned in the stream so that the following requirements are met:

- The flow velocity at the intake to the sampler shall be representative of the velocity in the part of the cross section of the river.
- Disturbance to the streamlines of flow shall be minimal.
- The intake of the sampler shall be correctly oriented vertically and horizontally.
- The nozzle of sampler shall be positioned against river flow.
- The air release nozzle shall be positioned upside so that air is released smoothly.

7.2.3 Sampling Location

The sampling location shall be project specific. It shall be selected through site visit by the hydrologist and/or sedimentologist in the early stages of planning.

The sampling location shall preferably be selected around headworks site. While finalizing the location, due consideration shall be given on representative location, spots where good mixing of sediment and water may be found.

7.2.4 Sampling Frequency

During the monsoon period, one or two samples shall be collected per day for regular sampling. The collection shall be done in the morning and in the evening. Every effort shall be made to capture flood samples.

During the pre-monsoon period (May – June) and the post-monsoon period (September – November), one sample per week shall be collected. Likewise, between November and May, one sample in the middle of each month shall be collected for the purpose of long term data collection.

8. LABORATORY ANALYSIS

As far as practicable, sediment samples from the river shall be analyzed on site to prevent errors due to falsification of samples during transport. This may also be desirable for ensuring that an adequate number of samples are analyzed, especially during monsoon, in a short time.

8.1 Laboratory Tests

Laboratory analysis of sediments shall primarily consist of the following:

a. Sediment concentration analysis.
b. Particle size distribution (PSD).
c. Mineralogical analysis.
d. Organic content analysis.

Concentration of sediments may be evaluated with the filtration method or the evaporation method. The filtration method may best be used on samples ranging up to about 10,000 mg/l of sediment that is mostly sand and about 200 mg/l of sediment that is mostly clay. For higher concentrations, the evaporation method may be used.

The PSD of suspended sediments shall generally be obtained using a combination of methods. To provide representative data for a range of the many conditions occurring throughout the year, a sufficient number of samples shall be analyzed for PSD. To study the behavior of sediment particles in different environments, analyses for PSD of fine particles in both native and dispersed settling media shall be analyzed.

For PSD, mineralogical analysis and organic content analysis, sufficient amounts of samples shall be available. For this, amounts of samples in excess of those required for regular sampling shall be collected.

8.2 Data Processing and Documentation

Results of the laboratory analysis shall be processed and documented in accordance with established standards. The results obtained from a set of sediment samples shall include:

- Distribution of suspended sediment load across the river section.
- Particle size and grading curves.
- Specific gravity of particles.

9. ESTIMATION OF SEDIMENT LOAD

Estimation of sediment loads shall entail prediction of suspended loads and bed loads. This shall be achieved through methods discussed in the following sections.

9.1 Suspended Load Estimation

Estimation of suspended load shall consist of predicting its concentration and distribution.

9.1.1 Suspended Sediment Load

Suspended sediment load shall be computed from the results of the sediment concentration analysis conducted on suspended sediment samples. The results of daily sampling expressed as the average daily sediment load in metric tons per day can be used to compute the average annual yield of sediment at a given site in metric tons per year.

To predict the suspended sediment load, a correlation between the sediment concentration in parts per million (ppm) or in metric tones/day and the water discharge in m^3/s shall be established. This correlation shall be established in the form of a sediment rating curve by plotting the sediment discharge as the abscissa and the water discharge as the ordinate on logarithmic paper and drawing a line of best fit through the plotted points. This curve shall then be combined with the flow duration curve drawn from stream flow records to estimate, with a fair degree of accuracy, the long-term sediment rate at a given site. To account for the

Using this analysis, the total sediment load at a given may be computed by accounting for the unmeasured portions of the total load, i.e. the bed load and the load passing in the bottom unsampled zone, usually about 7.5 to 15.0 cm from the stream bed which cannot be sampled by the sampler. This may be done through some correlation. For wide shallow streams, the correlation may be significant. Where judgment shows a small error due to the unsampled load, a correction of 5 to 10 percent may be added to account for this.

9.1.2 Suspended Load Distribution

The vertical distribution of suspended load may be estimated from the following function developed by Rouse (Sir M. MacDonald and Partners, 1990):

Eq. 1

$$\frac{C}{C_a} = \left[\frac{a(d-y)}{y(d-a)}\right]^{z}$$

where C is the concentration at elevation y above the bed, C_a is the concentration at a reference elevation a above the bed, d is the flow depth w is fall velocity and z is the Rouse number given by

Eq. 2

$$z = \frac{w}{\beta k V_*}$$

in which β is the ratio of sediment diffusion coefficient to momentum coefficient (assumed 1.0), K is the Von Karman constant (equal to 0.4) and V_* is the shear velocity equal to

Eq. 3

$$V_* = \sqrt{gdi}$$

In the above equation, g is the acceleration due to gravity and i is the hydraulic gradient.

9.2 Bed Load Estimation

Bed load may be estimated using one of the following methods:

a. Stream sampling.

b. Analytical or empirical methods.

Stream sampling may be performed using different types of samplers, such as the box-type sampler, the slot-type sampler, etc. As these samplers do not yield satisfactory results, bed loads may generally by taken as a certain percentage of the suspended material. Depending upon the nature of the bed material, etc., these percentages may vary from 3 to 25 percent of the total suspended load. Normally, bed load may be adopted as 10 percent of the total suspended load. For minor projects, some indirect methods, such as the Modified Einstein method, may be used to calculate the bed load.

9.2.1 Estimation Based on Measured Suspended Load

Bed load may be estimated as a percentage of the suspended load using the figures proposed by Maddock and listed in shown in Table 1.

Table 1: Maddock's classification for determining bed load based on suspended load

Concentration of suspended sediment (ppm)	Type of material forming channel of stream	Texture of suspended material	Bed load in terms of suspended load (%)
< 1000	Sand	Similar to bed material	25 – 150
< 1000	Gravel, rock, or consolidated clay	Small amount of sand	5 -12
1000 – 7500	Sand	Similar to bed material	10 – 35
1000 – 7500	Gravel, rock, or consolidated clay	25% sand or less	5 -12
> 7500	Sand	Similar to bed material	5 – 15
> 7500	Gravel, rock, or consolidated clay	25% sand or less	2 – 8

(Source: (Sir M. MacDonald and Partners, 1990))

9.2.2 Estimation Based on Empirical Formulae

Bed loads may be estimated using empirical relations of the following general forms:

Eq. 4 $$q_b = f(\tau - \tau_c)$$

Eq. 5 $$q_b = f(q - q_c)$$

Eq. 6 $$q_b = f(V - V_c)$$

Eq. 7 $$q_b = f(\varpi - \varpi_c)$$

In these equations, q_b is the bed load discharge per unit width, τ is the tractive force, q is the water discharge per unit width, V is the mean flow velocity and ϖ is the stream power per unit bed area. The subscript c in the equations signifies critical values for incipient motion.

For bed load prediction, the empirical equations given in the following sections may be used.

9.2.2.1 Dubois's Equation

The bed load q_b, in m^3/s/m, may be computed using Dubois's equation (Maidment, 1992)

Eq. 8 $$q_b = k_b \tau(\tau - \tau_c)$$

where k_b is a constant depending upon the grain size (d in mm), τ is the bed shear stress in N/m^2 that is fully responsible for the movement of bed load and τ_c is the critical bed shear stress in N/m^2 which is the minimum required to move the grains. For use in Eq. 8, k_b, τ and τ_c shall be computed from the following relations:

Eq. 9 $$k_b = \frac{0.001798}{d^{0.75}}$$

Eq. 10 $$\tau = 0.97\gamma_w RS$$

Eq. 11 $$\tau_c = 0.765d$$

where d is the grain size in mm, γ_w is the specific weight of water in N/m^3, R is the hydraulic radius of the river in m and S is slope of the riverbed.

9.2.2.2 Schoklitsch's Equation

Using the Schoklitsch equation (Maidment, 1992), the unit bed load q_b in kg/s/m shall be computed as

Eq. 12 $$q_b = \frac{7000S^{1.5}}{D_{50}}\left(q - q_c\right)$$

where S is slope of the water surface or river bed, D_{50} is the median size of bed material in mm, q is the unit water discharge in $m^3/s/m$ and q_c is the critical unit discharge at which bed load transport begins in m3/s/m. q_c may be determined by the following equation suggested by Tilrem:

Eq. 13 $$q_c = 1.94 \times 10 - 5D_{50} / S^{4/3}$$

D_{50} used in Tilrem's equation shall be representative of the surface material on the riverbed, while the D_{50} used in the Schoklitsch equation shall correspond to the values derived from the sub-surface samples.

The Schoklitsch relationship may also be used to compute the amount of gravel moved within different size classes, and these sub-totals shall be accumulated to arrive at the total gravel load.

9.2.2.3 Goncharov's Equation

Using Goncharov's equation (Maidment, 1992) the unit bed load q_b in kg/s/m shall be computed as

Eq. 14 $$q_b = 2.08\left(\frac{V}{V_c}\right)^3\left(\frac{d}{h}\right)^{0.1}\left(V - V_c\right)$$

where V_c is the critical flow velocity in m/s, V is the mean flow velocity in m/s, d is the mean grain diameter in m and h is the mean depth of flow in m. V_c shall be obtained as

Eq. 15 $$V_c = k\sqrt{d}$$

where k depends on the units of V_c and d. For d in mm and V_c in m/s, values of k may range between 2 and 12 for various particle weights, types of flow and friction conditions.

10. PREDICTION OF SEDIMENT YIELD

As adequate sediment data is generally not available in the majority of river basins in Nepal, prediction of sediment yield shall be based on catchment yield. This shall be achieved using the following methods:

a. Himalayan Sediment Yield technique (Galay, 1987)
b. Equations proposed for Indian catchments.

In addition, Zamarin's sediment transport formula (Nayak, 1993) may be used.

10.1 Himalayan Sediment Yield (HSY) Technique

Using the Himalayan Sediment Yield technique (Galay, 1997) that has been developed based on a comprehensive study of sediment yield in major river basins of Nepal, an estimate of the total basin sediment yield S_y in tons/year shall be obtained as

Eq. 16
$$S_y = Z_{sy} A$$

where Z_{sy} is the zone specific yield in tons/sq. km/year and A is the zone catchment area in sq. km. For this estimation, Z_{sy} shall be obtained for the concerned physiographic region from Table 2.

Table 2: Physiographic zones and their respective yields

Physiographic zone	Elevation (m)	Specific sediment yield (t/km²/yr)
High himalayas	Above 6,000	500
High mountains	3,500 ~ 6,000	2,500
Middle mountains	1,000 ~ 3,500	5,000
Siwaliks	500 ~ 1,000	7,500

(Source: Galay, 1987)

10.2 Equations Developed for Indian Catchments

When observed data on suspended sediments is not available, empirical relations developed for Indian catchments may be used to predict the annual sediment yield. For a catchment of area A in sq. km, the annual sediment yield Y_s in m^3 may be determined using the following equations developed for north Indian catchments (IOE, 1995):

Eq. 17
$$Y_s = \begin{cases} 3950A^{0.689} & \text{for } A > 150 \text{ sq. km.} \\ 15340A^{0.736} & \text{for } A < 150 \text{ sq. km.} \end{cases}$$

Other formulae that may be used for this purpose are Joglekar's general formula (IOE, 1995) given by

Eq. 18
$$Y_s = 5970A^{0.76}$$

or Khosla's formula (IOE, 1995) based on silting of Indian reservoirs and written as

Eq. 20
$$Y_s = 3230A^{0.72}$$

10.3 Zamarin's Formula

Zamarin's formula (Nayak, 1993) shall be used to estimate the suspended sediment transported by unit volume of flow ρ_{tr}, in kg/m^3, as

Eq. 19
$$\rho_{tr} = 0.022\left(\frac{V}{\varpi_0}\right)^{1.5}\sqrt{RS}$$

where V is the mean velocity of flow in m/s, ϖ_0 is mean settling velocity in m/s, R is hydraulic mean radius in m and S is the hydraulic gradient of the flow.

Eq. 17 is applicable for $0.002 \leq \varpi_0 \leq 0.008$ m/s. For finer particles of sediments with ϖ_0 between 0.0004 and 0.002 m/s, the formula may be used as

Eq. 16
$$\rho_{tr} = 11V\sqrt{\frac{RSV}{\varpi_0}}$$

11. DOCUMENTATION

Upon their completion, sedimentological investigations shall be properly documented. The documentation shall include, but not be limited to, the following:

- Description of the catchment, including its area, slope, soil type, vegetation, altitude, geology, hydrology; glacier processes, population pressure, agriculture practice, deforestation and other factors affecting sediment transport rate in the catchment.
- Availability of sediment data and their quality.
- Field observations at the proposed site.
- Extension and recovery of sediment data.
- Analysis of sediment data.
- Prediction of bed load and suspended load.
- Laboratory analysis of particle size and mineralogy.
- Results and data presentation (sediment delivery ratio, sediment rating curve).

PART 1D – GEOLOGICAL AND GEOTECHNICAL INVESTIGATIONS

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Part

1D

Geological and Geotechnical Investigations

1. PURPOSE

Part 1D of the *Design Guidelines for Headworks for Hydropower Projects* defines minimum requirements and provides procedural guidelines for geological and geotechnical investigations performed in support of engineering and design of headworks of run-of-river hydropower projects. These guidelines aim at ensuring consistency in the nature and extent of such investigations conducted for headworks development in Nepal.

In recognition of the wide variations in geological conditions from one site to another, the guidelines do not attempt to set forth standardized investigation programs for a given set of geological conditions or a particular headworks structure. It is anticipated that the agency responsible for the development of the headworks will tailor its investigation programs to the particular headworks needs and local conditions in accordance with standard practice, judgment and experience, satisfying the requirements and expectations of these guidelines.

2. SCOPE

The guidelines focus on the desirable quality and quantity of geological and geotechnical investigations required for design of headworks. They discuss the planning considerations and principal methods of investigation for acquiring the data and information on the geological features and geotechnical design parameters of the headworks area. They also define standard procedures for the documentation and reporting of these investigations.

3. TERMINOLOGY

Terms used in these guidelines are defined below:

Adit Temporary underground passage from the surface into an underground location for access and investigation.

Aquifer Body of rock that is sufficiently permeable to conduct ground water and to yield significant quantities of water to wells and springs.

Bedding	Arrangement of a sedimentary rock in layers; stratification.
Bedding plane	Division plane in sedimentary or stratified rocks that separates each successive layer or bed from the one above or below.
Bedrock	Solid rock that underlies gravel, soil or other superficial material.
Coarse aggregates	Aggregates comprising materials graded from 4.75 mm to 75 mm in size or any size or range of sizes within such limits.
Dip	Maximum inclination of a structural discontinuity plane to the horizontal.
Dip direction	Direction of the horizontal trace of the line of dip.
Discontinuity	Surface separating two unrelated groups of rocks.
Fault	Fracture or fracture zone along which there has been displacement of the sides relative to one another parallel to the fracture.
Fine aggregates	Aggregates with materials not larger than 4.75 mm in size.
Fold	Bend or plication in bedding, foliation, cleavage or other planer features in rocks.
Joint	Surface of fracture or parting in a rock, without displacement; the surface of which is often plane and may occur with parallel joints to form a joint set.
Overburden	Loose or consolidated rock material that overlies bedrock and must be removed prior to excavation.
Permeability	Capacity of a porous rock, sediment or soil for transmitting a fluid.
Porosity	Ratio of the aggregate volume of interstices in a rock or soil to its total volume, usually stated as a percentage.
Riprap	Layer of large, durable rock fragments to preserve the shape of a surface slope or underlying structure by preventing erosion due to wave action or stream current.
Rock	Aggregate of one or more minerals or a body of undifferentiated mineral matter or of solid organic material.
Rockfill	Embankment constructed of rock fragments in portions of earthfill dams or in rockfill dams.
Sand	Detrital particle smaller than a granule and larger than a silt grain, having a diameter in the range of 1/16 to 2 mm.
Strike	Geographic direction of intersection of the bedding plane with a horizontal plane.

4. OBJECTIVE OF INVESTIGATIONS

Geological and geotechnical investigations conducted for design of headworks shall lead to an appraisal of the geological and geotechnical conditions in the headworks area so that the influence of these conditions on the design, construction and operation of the headworks, including its safety and economy, is as fully understood as possible. For this purpose, the investigations shall aim at determining the following:

a. General geological setting of the headworks area and its vicinity.
b. Geological conditions related to site selection.
c. Characteristics of foundation rocks and soils.
d. Geotechnical design parameters for the headworks.
e. Availability of suitable construction material.

f. Any other geologic conditions that may influence design, construction and long term operation of the headworks.

5. SCOPE OF INVESTIGATIONS

To attain the objectives listed in Section 4, the investigations listed below shall normally be performed in the headworks area of run-of-river hydropower projects:

a. Surface investigations.
b. Subsurface investigations.
c. Large-scale, prototype investigations.
d. Laboratory investigations.
e. Construction material investigations.

For a particular headworks development, the need and scope of each type of investigation shall be defined through the planning procedures suggested in Section 6. The investigations thus planned shall be conducted using the procedures described in the subsequent sections.

6. INVESTIGATION PLANNING

Geological and geotechnical investigations shall be properly planned to establish their scope, extent, procedures, instruments and manpower requirements. The planning shall be accomplished through desk studies and site visits conducted jointly by the headworks design team and the investigation team.

6.1 Planning Considerations

Geological and geotechnical investigation programs for headworks shall be prepared taking into account the planning and design requirements, complexity of local geology, nature of structures, available data, economy and schedule.

Data Requirements for Planning and Design

Geological and geotechnical investigations shall be designed to collect data and information that are adequate and appropriate for the planning and design of the headworks. The investigations shall be conducted in sufficient detail to permit evaluations, analyses, design, quantity and cost estimation, construction planning and preparation of specifications based on site-specific information and data, with minimum use of general assumptions on geological conditions and geotechnical strength parameters.

In order to achieve the above objective, the nature and extent of data and information to be collected shall be established considering site geological and geotechnical factors that could critically affect the design, construction and safe performance of the headworks structures. These factors shall include, but not be limited to, the following:

- Local and regional geological structures, such as folds, faults, shear zones, bedding, schistosity, foliations, cleavage, joints, seams, underground cavities and channels, etc.;
- Lithology, including types, locations, sequence, thickness, areal extent of rocks and soils, anisotropy, mineral and chemical composition, etc.;
- Physical condition of various rock and soil strata, including their attitude, degree of weathering, shattering, cracking and alteration, infilling, etc.;
- Hydrogeology, including depth and direction of flow of ground water, type of water table, depth of and pressure in artesian zones, etc.;
- Engineering and index properties of the overburden and rocks, such as compressive strength, shear strength, compressibility, deformability, water tightness, permeability, etc.

Sufficiency of data and information shall further be ensured by adequately distributing the investigations over the headworks area. The investigations shall cover possible locations of diversion structures, spillways, embankments, intakes, settling basins, appurtenances and borrow sites. For diurnal pondage run-of-river projects, they shall also cover the reservoir rims.

Complexity of Local Geology

The geological and geotechnical investigations shall be planned considering the complexity of the local geology. Sites with complex geology shall be investigated in sufficient detail to enable the designer to make an informed decision on the suitability of the sites for hosting the proposed headworks structures and to design cost-effective remedial measures. For this purpose, the extent of investigations shall be decided through proper identification of site geological conditions that could introduce uncertainties or risks in the design of the headworks or could lead to unfavorable or hazardous situations during their construction and operation. Such conditions could result from the presence of the following geological features at the site:

- Complex folding and faulting.
- Axial regions of folded sequences.
- Upstream limbs of synclinal bends.
- Faults and shear zones in foundation beds.
- Highly jointed, fractured or shattered rocks.
- Unconformities, overlaps, etc.
- Highly complex lithological variations, with alternating layers of a number of rock types of variable composition.
- Paleo-channels in river beds.

Natural Hazards

Investigations plans shall be capable of generating an adequate basis for assessing potential geological hazards at the headworks site, evaluating the impacts of these hazards on the headworks safety and designing suitable risk mitigation or management measures against them. In view of Nepal's high seismicity, investigation of seismic hazards at the headworks site shall be mandatory. Hazards from glacial lake outburst floods (GLOF) or other active natural geological forces shall also be investigated thoroughly if such hazards exist at the headworks site.

Nature of Headworks Structures

The type of geological and geotechnical investigations shall correspond to the potential modes of failure and construction material requirements of diversion and other headworks structures. For concrete gravity dams, the investigations shall concentrate on determining the foundation bearing and shearing strengths, potential for uplift and differential settlement of the foundation material and the availability of the quality aggregates in the vicinity of the headworks. Likewise, the investigations for embankments shall focus on establishing the foundation bearing capacity and the availability of the required dam materials, such as core and filter materials, close to the dam site.

The extent of investigations shall be commensurate with the size, importance and hazard potential of the headworks. Thorough investigations shall be made for headworks which are either large in magnitude or are a key part of the regional or national electrical system. Headworks with significant or high hazard potentials, whose failure could pose serious threat to life and property, shall also be comprehensively investigated.

Economy in Construction

To ensure economy in construction, prospecting for construction materials shall be planned systematically to identify suitable quarry sites or borrow areas that are within an easy haulage distance of the headworks. For this purpose, the prospecting shall start from the dam site and extend outward in all directions to guarantee that nearby quarry sites or borrow areas are not overlooked. Generally, the range of investigations from the headworks site shall be limited economically to about 20 km for cohesive material, sand and gravel and between 10 to 15 km for rocks, and the nearest quarry sites or borrow areas shall be selected.

Available Data

To avoid duplication of efforts and generation of redundant data, the investigations shall be planned to acquire information that is not already available for the headworks area from previous investigations. The need, nature and extent of further investigations at a particular location shall be decided based on an assessment of the geological conditions and geotechnical properties of that area from available information. While doing so, care shall be exercised to assess the quality and reliability of such information.

Protection of Environment

During investigation planning, suitable measures aimed at protecting the environment of the headworks areas and its surroundings shall be designed. Access routes to the investigation sites shall be selected with care to minimize damage to the environment. Suitable measures shall be devised to control the operation of investigation equipment at all times and to hold the damaged areas to the minimum, consistent with the requirements for obtaining adequate data. Plans for keeping sediment flow from the investigation sites into water bodies within permissible levels shall be prepared in accordance with specified environmental regulations. Steps to restore and rehabilitate areas disturbed by the investigations to a natural appearance after the investigations shall also be planned.

Economy and Schedule of Investigation

The investigation program shall be designed to fulfill its objectives in an economical and timely manner. For this purpose, the investigations shall be directed towards obtaining only such information as may be important for the development and safe performance of the headworks. The program shall not attempt to develop extraneous information. To ensure that resources are not spent on investigating unsuitable sites, the program shall prescribe a sequence of investigations so that the most questionable and critical areas are explored first.

6.2 Planning Data and Information

At any stage of headworks development, geological and geotechnical investigations shall be planned based on an assessment of site conditions vis-à-vis the planning and design needs of the proposed headworks development. This assessment shall be performed through desk studies of available information on the proposed headworks, topography, geology, hydro-geology and seismo-tectonics of the headworks area. The information thus collected shall be substantiated and/or upgraded through site visits.

Information on Proposed Headworks Development

Information on the proposed headworks development plans shall be obtained from reports of previous studies. Depending on the stage of development of the headworks, such studies could include master plan, inventory, reconnaissance, pre-feasibility or feasibility studies conducted by different agencies. Information obtained from these reports shall be discussed and confirmed with the headworks design team.

Information on Topography

For investigation planning, information of geological and geotechnical significance shall be inferred from the topography of the headworks area. In particular, topographic features such as landforms, drainage patterns, slopes, prominent springs and wet areas, landslides and related features, quarries, etc. shall be carefully studied to deduce information on faults, soil cover, general soil and rock types, rock structures, geomorphic history, borrow areas, etc. present in the area. For instance, faults may be estimated and forecast by tracing continuous linear river systems, continuation of waterfalls or steep cliffs, sudden changes in ridgeline or slope transition line or by identifying cuts in quaternary deposits like river terraces. Similarly, the existence of hard and soft rocks or soils may be estimated from the presence of steep and gentle terrain, respectively. Very gentle topography may be treated as an indication of the presence of river deposits in the form of river terrace of different geological ages.

Topographical information required for drawing geological inferences shall be derived from available large and small-scale topographic maps. These maps shall include engineering site plans prepared as part of previous studies on the headworks and the following maps published by the Survey Department, GoN:

a. 1:125,000 scale district topographic maps with a 250 m contour interval.
b. 1:50,000 scale topographic maps with a 40 m contour interval.
c. 1:25,000 scale topographic maps with a 10 / 20 m contour interval.

As a general rule, interpretation of topographic maps shall proceed from small-scale maps through intermediate-scale maps to large-scale maps as the geologic investigation proceeds from the general to the specific.

Information on geology and geomorphology of the headworks area shall also be deciphered from its topography through the interpretation of aerial photographs and land satellite imageries. For this purpose, aerial photographs, available at 1:40,000 or larger scale, and satellite imageries available at a scale of 1:250,000 shall be obtained from the Survey Department, GoN. Aerial photographs and satellite imageries for several hydropower projects studied in the past may also be availed from the Water and Energy Commission (WEC), Ministry of Water Resources (MoWR), GoN.

Information on Regional Geology

For planning purposes, information on the regional geology, such as formation descriptions, formation contacts, structure, fault locations, etc., shall be extracted from available regional geological maps and reports on the headworks area. In particular, the following geological maps published by the Department of Mines and Geology, GoN, shall be referenced:

a. 1:1,000,000 scale compiled geological map of Nepal;
b. 1:50,000 and 1:63,360 scale maps of different regions of Nepal;
c. 1:25,000 and 1:250,000 scale maps available for selected areas of Nepal.

Regional geological information may also be assessed through remote sensing information. In particular, aerial photographs and satellite imageries shall be used for large-scale regional interpretation of geologic structure, analyses of regional lineaments, drainage patterns, rock types, soil characteristics, erosion features and availability of construction materials. They may also be used to recognize geologic hazards such as faults, fracture patterns, subsidence, and sink holes or slump topography.

Information on Site-specific Geology

At the investigation planning stage, site-specific geological information shall be derived from a study of the regional geology and available reports. Based on this information, a geological map of the headworks area shall be prepared at a suitable scale to provide initial insight into the geology of the area and to facilitate selection of the main and alternate dam sites.

Wherever possible, site-specific geological information, such as local faults, orientations of joints, detailed lithological descriptions, details on depth to rock, etc., shall be obtained from available large-scale geological maps. This information may be obtained from reports and maps on hydropower projects studied at or near the proposed headworks location. Such reports and maps are available for projects studied up to master planning, reconnaissance, pre-feasibility, feasibility or design levels by the Nepal Electricity Authority (NEA), different agencies of MoWR, including DoED, WEC and the erstwhile Electricity Department. These documents may also be available for hydropower projects studied by private developers.

Information on Hydrogeology
Information on the hydrogeology of the headworks area, including surface drainage, ground water quality, ground water level contours, seepage patterns and aquifer locations and characteristics, shall be obtained from available engineering geological maps of the area. Such maps may be obtained from documents on ground water or irrigation projects studied by the Department of Irrigation, GoN, at or near the proposed headworks site. They may also be availed from reports on hydropower projects studied by NEA, DoED, WEC, etc. in the vicinity of the proposed headworks site.

Other Information Relevant to Headworks Area
Where needed, information on land use, mining, etc. in the headworks area shall also be collected. This information shall be obtained from the Survey Department or the DMG.

Information from Site Visits
Information collected from desk studies shall be confirmed, corrected or expanded through field studies conducted by a multidisciplinary team of engineering geologists, geotechnical engineers, design engineers and representatives of other disciplines as required. These studies shall include, but not be limited to, the following:

- Field checking of existing maps.
- Demarcation of rock and soil boundaries.
- Identification of rock and soil types, degree of weathering, discontinuities, etc.
- Assessment of slopes in and around the proposed site.
- Assessment of ground water levels.
- Assessment of the accessibility, general quality and approximate quantity of construction material in the vicinity of the proposed site.
- Assessment of land instabilities close to the headworks area, if any.
- Assessment of site accessibility, ground conditions and right-of-entry problems that could affect field exploration work.
- Identification of cultural features, such as power lines, pipelines, access routes, historical and archaeological sites, etc., that could affect exploration work and site location.

Depending on the stage of investigations being planned and the uncertainties involved, the field studies shall be facilitated by preliminary geological mapping and basic field tests such as test pitting, trenching, rebound hammer tests, penetrometer tests, selective geophysical surveys, etc. The findings of these studies shall be documented through sketches, notes and photographs.

7. SURFACE INVESTIGATIONS

Surface investigations shall be performed to study the geology of the headworks area using field operations that do not disturb the ground significantly during their performance. These operations shall include:

a. Geological field mapping.
b. Rock mass classification.
c. Surface geophysical investigations.

Surface investigations shall be conducted at preliminary stages of headworks development to collect generalized geological information on the headworks area. At advanced stages of development, the extent and precision of these investigations shall vary with the application and purpose for which the geological information is to be used.

7.1 Geological Field Mapping

Geological field mapping shall be performed to generate a graphical representation of the geological conditions of the headworks area or parts thereof. The mapping shall consist of the following activities:

a. Areal mapping.
b. Site mapping.
c. Discontinuity mapping.

7.1.1 Areal Mapping

Areal mapping shall be performed in the headworks area to prepare an accurate picture of its geological framework, including the spatial distribution and position of its stratigraphic units, its structural features and its surface forms. The resulting map shall generate sufficient geological information to permit selection of suitable sites for the headworks and to prepare appropriate geological and geotechnical investigation schemes for the selected sites.

7.1.1.1 Coverage

The areal extent of the mapping shall depend on the type and size of the headworks and the complexity of the regional geology. In general, the mapped area shall include the headworks site and its surrounding areas that could influence, or be influenced by, the headworks.

7.1.1.2 Mapping Procedure

Areal mapping shall begin with the incorporation of available geological information on the headworks area into available topographical or geological maps of the area. The information shall include those identified and collected during investigation planning through available maps and reports, remote sensing techniques and site visits. Any additional geological data generated on the area since then shall also be included.

Field mapping of the headworks area shall commence following the preparatory works. For this, the headworks and its adjacent areas, in particular the major and minor streams in the area, shall be traversed to collect information on geological features of the area including, but not limited to, the following:

a. Types of surface deposits and their genetic origins.
b. Continuity of rock and formation.
c. Rock properties, particularly the distribution, thickness and hardness of soft rock and non-solidified materials.
d. Location, scale, width and attitude of major and minor faults.
e. Bedding and foliation planes.
f. Prominent and random joint sets, including their persistency.
g. Weak zones such as sheared and fractured zones.
h. Axial traces of folds.
i. Degree of weathering of rocks.
j. Karst topography or other features that indicate high leakage potential.
k. Water well levels, springs, surface water, water-sensitive vegetation or other evidence of the ground water regime.

l. Soluble or swelling rocks such as gypsum or anhydrite.
m. Potential landslide areas.
n. Potential borrow and quarry areas.
o. Reservoir shoreline erosion potential.
p. Landfills, dumps, surface impoundments and other potential environmental hazards.
q. GLOF.

Information collected at the site shall be documented through geological sketches and notes. The main geological features of the site shall also be photographed from different angles.

7.1.1.3 Detail of Mapping

Areal mapping at the detailed design stage shall be a continuation of the mapping conducted till the feasibility stage. More details on the feasibility level areal maps shall be added from information obtained from inspections of test pits, trenches, etc. and results of other subsurface explorations.

7.1.1.4 Analysis and Documentation

The information collected during areal mapping shall be compiled and analyzed to prepare geological maps of the headworks area at desired scale. These maps shall show all pertinent geological conditions of the area including landslides, sinkholes, potential leakage areas, etc. Geological profiles at suitable locations of the headworks areas shall also be developed.

Geological maps and sections prepared at feasibility level investigation shall be revised and updated as additional data becomes available from surface and subsurface explorations. To facilitate this process, the maps may be prepared in digital format using computer-aided designing and drafting (CADD) packages. Depending on the size of the headworks and availability of funds, the use of a Geographic Information System (GIS) may be also considered for this purpose.

7.1.1.5 Mapping Scale

The recommended scales for areal mapping at different stages of headworks development are listed in Table 1. Where a range of scales is provided, the scale shall be chosen from the scale of topographic maps or engineering site plans available at that stage of development.

Table 1: Recommended scales for areal mapping

Purpose of survey	Target map scale
General reconnaissance	1:25,000 or 1:50,000
Pre-feasibility study	1:1,000
Feasibility study	1:500 – 1:1,000
Detailed design	1:100 – 1:500

(Source: IS: 6065 Part 1 – 1985)

7.1.2 Site Mapping

Site mapping shall be performed to prepare large-scale, detailed engineering geological maps for each site identified or selected for the various structures. The mapping shall generate sufficiently detailed information to permit selection of the most suitable site among candidate sites for the headworks structures, to prepare detailed geological and geotechnical investigation plans for them and to prepare their final designs, plans and specifications.

7.1.2.1 Coverage

The extent of site mapping shall depend on the type of headworks structures proposed. For concrete dams, the mapping shall cover an area at least equal to twice the height of the dam towards the upstream and downstream directions of the area covered by the main dam foundation. For embankment dams, the mapping shall extend along the river to a distance of about four times the height of dam both on the upstream and downstream sides of the proposed dam axis. Irrespective of the type of dam, these surveys shall extend up to 100 m above the top of the dam in areas of immature topography and up to 25 m above the top of dam in mature topography. At dam sites involving special geological problems, such as the instability of hill slopes, the mapping shall extend to cover such areas also.

7.1.2.2 Mapping Procedure

Site mapping shall commence with the incorporation of available geological information, including that obtained from areal mapping, into topographical maps of desired scales. If a significant gap in time exists between the areal and site mappings, geological information generated, compiled, analyzed and published by other agencies shall also be integrated into the maps. In such an event, the new information, particularly those related to seismology and hydrogeology, shall be correlated with the previously collected information for evidence of significant changes in the geological knowledge of the study region.

Field investigations conducted after preparation of the initial site geological maps shall focus on site-specific and detailed mapping of geological features listed in Section 7.1.1.2 so that the site geological structure, lithology, stratigraphy and hazards are as precisely understood as possible. In addition, the rock quality designation for each site shall be estimated using techniques described in Section 7.2.2.

During the field investigations, the geological information collected shall be recorded in detail through sketches and notes. The principal geological features of the site shall also be documented through photographs.

7.1.2.3 Detail of Mapping

Site mapping during the detailed design stage shall focus on further detailing of sites selected among the candidate sites. It shall be a continuation of the site mapping performed at the feasibility stage, with more details being supported by surface and subsurface investigations as well as rock and soil tests performed to provide geotechnical design parameters.

7.1.2.4 Analysis and Documentation

Geological and geotechnical information collected during site mapping shall be compiled and analyzed to prepare geological maps of the candidate or selected sites. These maps shall exhibit information on all pertinent geological features of the sites available at the particular stage of investigation, including the following:

- Boundaries of geologically different units of overburden and rock.
- Expected variation in soil and rock properties as gathered from tested samples.
- Results of geophysical studies in different rock and overburden formational units, such as longitudinal wave velocity, Poisson's ratio, Young's modulus and electrical resistivity.

In addition to plans, geological profiles of the different sites shall be developed. The profiles shall preferably be prepared along and across the axes of the proposed structures or along lines that yield useful insight into the site geological conditions.

To develop a complete understanding of the geological site conditions, the site geological maps and profiles shall be progressively refined using information obtained from subsurface investigations, in-situ and laboratory tests and large-scale prototype tests. For this purpose, the maps and profiles shall preferably be prepared in digital format using CADD packages in conjunction with suitable database programs. For larger projects, use of GIS shall also be considered to facilitate map correction and refinement based on new information.

7.1.2.5 Mapping Scale

The scales recommended for site mapping at different stages of headworks development are listed in Table 2. Generally, the largest available scale topographic maps shall be chosen for site mapping.

Table 2: Recommended scales for site mapping

Purpose of survey	Target map scale
Feasibility study	1:200 – 1:1,000
Detailed design	1:100 – 1:500

(Source: IS: 6065 Part 1 – 1985)

7.1.3 Detailed Discontinuity Mapping

Detailed discontinuity mapping shall be performed at sites identified or selected for major headworks structures and at sites of potential land instability around them, including those in the reservoir areas of diurnal pondage run-of-river hydropower projects. The mapping shall yield sufficient information on the characteristics of the discontinuities present at these sites to enable structural stability analyses and support designs to be carried out.

7.1.3.1 Coverage

Discontinuity mapping shall generally cover the areas identified in Section 7.1.2.1. However, the mapping shall extent beyond these areas if there is a likelihood of discontinuities in the adjoining areas influencing the stability and support design.

For studying land instability, detailed discontinuity mapping shall spread over an area that is adequate to identify all possible mechanisms influencing the suspected instability. This area shall be identified through studies of areal and site maps prepared for the headworks site.

7.1.3.2 Mapping Procedure

Field investigations for detailed discontinuity mapping shall aim at collecting information on the following characteristics of joints, bedding planes, faults, foliations and cleavages present in the area of study:

- Orientation (dips and dip directions).
- Spacing.
- Length.
- Width/openness.
- Roughness.
- Wall weathering.
- Infilling.

Depending on the parameter under consideration, qualitative descriptions or quantitative measurements shall be noted. Qualitative descriptors shall be based on the rock mass classification proposed to be used in the investigations (refer Section 7.2).

Information on the discontinuities shall be documented through field sketches and notes. Where needed, photographs shall be used to supplement the sketches and notes.

7.1.3.3 Extent of Mapping

In general, at least 30 measurements shall be taken for each set of joints and beddings/ foliations in one outcrop of rock. In critical areas, the number of measurements may reach up to, or even exceed, 200. To develop a reliable basis for statistical analyses, more than 100 joints shall generally be measured for each major headworks structure.

7.1.3.4 Analysis and Documentation

Statistical analysis of the data on the orientation of discontinuities shall be carried out to find out the average dip and dip directions of the main joint systems, i.e. the significant bedding planes, joints and other structural features, present in a rock mass at each site. This analysis shall be performed by plotting the data on stereonets manually or using computer softwares.

7.2 Rock Mass Classification

Rock mass classification shall be performed to arrive at a general rating of the rock mass quality in the headworks area. The classification shall quantify the site geological information and shall serve as a basis for estimation of preliminary support when subsurface geological information and results of laboratory and *in-situ* test are not available.

7.2.1 Rock Mass Classification Systems

Of the several rock mass classification systems available, the following systems shall be used for classifying the rock mass in the headworks area:

a. Rock Quality Designation Index.
b. Geomechanics Classification or RMR System.
c. Rock Tunnel Quality Index or Q System.

During early stages of investigations, at least two of these methods shall be used at each site to compare their results and thus build confidence and develop consistency in the use of these systems.

7.2.2 Rock Quality Designation Index

The rock quality designation (RQD) index shall be estimated from drilled cores (Deere and Deere, 1989). Where drilled cores are not available, the RQD shall be estimated from visible discontinuity traces in surface exposures or exploration adits (Palmstrom, 1982).

7.2.2.1 Deere and Deere's Method

Using Deere and Deere's method (1989), the RQD at a site shall be estimated through the relation

Eq. 1

$$RQD = \frac{\sum L_{sc}}{T_{lc}} \times 100$$

where L_{sc} is the length of sound core pieces equal to or longer than 100 mm and T_{lc} is the total core run length.

Core Sizes

RQD shall be estimated from NW (54.7 mm diameter) or higher size cores drilled with a double tube core barrel. Cores from NQ (47.6 mm diameter) or higher wire line sizes may also be used for this purpose. However, the use of BQ (36.5 mm) and BX (42.0 mm) cores for estimation of RQD shall be discouraged.

Measurement

To obtain a representative estimate of the rock mass quality, RQD shall be estimated for each core run. While making this estimate, the following precautions shall be observed:

a. Measurements shall be performed at the time the core is retrieved so that the effects of post-removal slaking and separation of core along bedding planes are avoided.
b. Core segment lengths shall be measured along the centerline or axis of the core.
c. Only sound segments of the core, i.e. segments without discolored or bleached grains or crystals, heavy staining, pitting, or weak grain boundaries, shall be measured.

d. Mechanical breaks in cores, i.e. breaks caused by drilling action or handling, shall be disregarded.

7.2.2.2 Palmstrom's Method

Using Palmstrom's method (1982), RQD shall be found from the number of discontinuities per unit volume. For clay-free rock mass, the following relationship shall be used:

Eq. 2 $$RQD = 115 - 3.3J_v$$

where J_v is the volumetric joint count, i.e. the sum of the number of joints per unit length for all joint (discontinuity) sets.

7.2.3 Geomechanics Classification or RMR System

The Geomechanics Classification System (Bieniawski, 1976) shall be used to classify rock mass in the headworks area by estimating its Rock Mass Rating (RMR). For this purpose, the rock mass shall be divided into a number of regions whose boundaries coincide with either a major structural feature, such as a fault or a change in rock type, or with significant changes in discontinuity spacing or characteristics within the same rock type. Thereafter, the RMR for these regions shall be estimated by summation of individual ratings for each of the following six parameters related to the rock mass:

a. Uniaxial compressive strength.
b. RQD.
c. Spacing of joints.
d. Conditions of discontinuities.
e. Ground water conditions.
f. Orientation of discontinuities.

Depending on the RMR thus estimated, the rock mass in each region shall be classified into different categories proposed by Bieniawski (1976).

Assignment of Individual Ratings

Ratings for the six parameters used to estimate the RMR shall be based on standard charts prepared by Bieniawski (1976) and published in standard textbooks on engineering geology or rock engineering. In order to assign these ratings, the uniaxial compressive strength shall be obtained from point load or uniaxial compressive strength tests on rock samples, the RQD shall be derived from the procedures described in Section 7.2.2 and the remaining parameters shall be noted from field observations.

Precautions

The RMR system shall preferably be used for classifying hard and jointed rock mass. Care shall be exercised in extending the use of this system to other types of rock masses.

7.2.3.1 Rock Tunneling Quality Index or Q System

Using the Rock Tunneling Quality Index System (Barton *et al*, 1974), classification of rock mass shall be performed through estimation of the Rock Tunnel Quality Index, or the Q value, from the relation

Eq. 3 $$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

where J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor and SRF is the stress reduction factor. Based on the value of Q thus obtained, the rock mass shall be classified into different categories proposed by Barton *et al* (1974).

Evaluation of Parameters
For computing the Q value, the RQD shall be derived from the procedures discussed in Section 7.2.2. The remaining parameters shall be obtained from field observations.

7.3 Surface Geophysical Investigations

Surface geophysical investigations shall be performed to make indirect measurements from the ground surface for rapid evaluation of the following aspects of the headworks area:

a. Subsurface geological conditions such as overburden thickness, degree of weathering, faults, shear zones, depth of sound bedrock, discontinuities, voids, ground water, etc.
b. Engineering properties, such as elastic moduli and densities, of the underlying strata.

Surface geophysical investigations shall generally be carried out in the early stages of field explorations, usually in combination with limited subsurface explorations. Results obtained from these investigations shall act a basis for finalizing locations and details of subsequent investigations.

7.3.1 Methods of Geophysical Investigations

Of the various methods of geophysical exploration available, the following shall be used for surface geological investigations of the headworks area:

a. Seismic refraction method.
b. Electrical resistivity method.

Surface geophysical investigations shall normally be performed using the seismic refraction method. However, the electrical resistivity method shall be preferred at sites with potential for water leakage and in areas likely to have cavities or significant changes in soil moisture content. In view of ambiguities inherent in geophysical methods, use of both methods in an integrated exploration program shall be considered.

7.3.2 Seismic Refraction Method

Seismic refraction surveys shall be carried out at the sites of all major headworks structures. These surveys shall be performed both along and across the axis of the structures. For dams, the surveys shall extend outwards on each side of the proposed dam foundation by at least 200 m.

7.3.2.1 Survey Procedure

Seismic profiles for refraction surveys shall be laid on the ground or water surface using appropriate survey techniques. Each profile shall be marked with pegs generally spaced at intervals of 5 m. To define the absolute location of the profile, some pegs placed along it shall be tied to the headworks control points.

Depending on site conditions, seismic waves shall be generated at the ground surface by explosives, hammering, dropping weights, rifle shots, harmonic oscillators or waterborne mechanisms. The travel time of refracted waves from different layers shall be recorded with a seismograph through geophones or hydrophones fixed on each peg along the profile.

7.3.2.2 Interpretation of Results

Data recorded during seismic refraction surveys shall be processed and suitably interpreted to deduce information on the subsurface geology. The thickness of different subsurface strata shall be estimated by plotting time-distance graphs. Likewise, the type of material in different layers shall be estimated by comparing recorded velocities with standard velocities.

Generally, a layer with velocity exceeding 2,000 m/s shall be considered to be bedrock while layers with lower velocities shall be taken as overburden material.

7.3.2.3 Precautions

Seismic refraction survey shall not be conducted in areas where a hard layer overlies a soft layer, in areas covered by concrete, asphalt pavement or other artificial hard crusts and in regions with frozen surface layer. Care shall also be exercised in interpreting results of seismic refraction in areas where a thin water-saturated zone or a weathered zone exists just above the bedrock.

7.3.3 Electrical Resistivity Method

The electrical resistivity method shall be adopted to estimate vertical and lateral variations in subsurface geological conditions in the headworks area. An estimate of vertical variations, which reflect stratification of the earth materials, shall be made through vertical profiling. Likewise, lateral variations, which are also indicative of local anomalous features such as soil lenses, isolated ore bodies, faults or cavities, shall be estimated through horizontal profiling. Occasionally, a combination of vertical and horizontal methods may be used.

7.3.3.1 Survey Procedure

Electrical resistivity survey shall be carried out by implanting two current electrodes and two potential electrodes in a selected array in the ground. An electrical current shall be applied between the current electrodes and the potential difference between the potential electrodes shall be measured. The current shall either be a direct current, a commutated direct current or an alternating current of low frequency.

During the surveys, the potential electrodes shall usually be kept in line between the current electrodes. In hilly terrains, the line of electrodes shall, as far as possible, be laid out along a contour. Where beds are known to dip steeply (more than about 10º), the line shall be laid out along the strike.

Vertical Profiling

For vertical profiling, the center point of the array shall be fixed, and the spacing between the electrodes shall be systematically varied. In general, a maximum electrode spacing of a least three times the depth of interest shall be used. The apparent resistivity curve from the survey shall be plotted as the survey progresses so that sufficient data is collected to define the asymptotic phases of the curves and any errors in readings or spurious resistivity values due to local effects are detected.

Where mapping of the depth to bedrock is desired, vertical profiling shall be done at each of a set of grid points. Before doing so, the results of resistivity surveys at a few stations shall be compared with drill hole logs to ensure that reliable quantitative interpretation of the resistivity can be made through this procedure.

Horizontal Profiling

Horizontal profiling shall be performed with constant electrode spacing by moving the array along a line of traverse or at points of a grid. The best electrode spacing for this purpose shall be first determined from vertical profiling and available geological information, such as the depth of the features of interest. The spacing between adjacent resistivity stations, or the fineness of the grid, shall be smaller than the width of the smallest feature to be detected or smaller than the required resolution in the location of lateral boundaries.

7.3.3.2 Analysis and Interpretation of Results

Vertical Profiling Data

Field data from vertical profiling shall be plotted to draw curves of apparent resistivity versus electrode spacing. These curves shall be interpreted to deduce the apparent resistivity values and thicknesses of the layer by qualitative comparison of observed response with that of idealized hypothetical models or on the basis of empirical methods. The deduced values shall be referred to the center point of the array

Horizontal Profiling Data

Apparent resistivity values obtained from field measurements for horizontal profiling shall be plotted as profiles or contoured on maps of the study area. Areas displaying anomalously high or low values, or anomalous patterns, of resistivity shall be identified from these plots.

Interpretation of horizontal profiling data shall be made qualitatively. It shall be guided by available knowledge of the local geology. Alternative interpretations shall be considered, and evidence from different independent sources shall be applied to the interpretation. For symmetrical arrays, resistivity value obtained shall be associated with the center of the array.

7.3.4 Calibration of Geophysical Measurements

To provide meaningful interpretation to them, geophysical measurements shall be calibrated against geological information obtained from boreholes or other subsurface explorations. This shall be achieved by associating zones of distinctive resistivity with specific soil or rock units on the basis of local field or borehole information.

8. SUBSURFACE INVESTIGATIONS

Subsurface investigations shall be carried out to gain geologic information below the natural ground surface in the headworks area. These investigations shall be conducted using surface invasive techniques, principal among which are the following:

a. Drilling.

b. Test pits and trenches.

c. Ground water studies.

d. Permeability testing.

e. In situ testing for determining geotechnical properties.

Subsurface investigations shall generally be performed to improve the accuracy of geological surveys. They may also be conducted to clear ambiguities and verify the results of geophysical surveys.

8.1 Drilling

Drilling shall be conducted at the locations of all major headworks structures primarily to define the geologic stratigraphy and structure of the site and to determine the depth of ground water. It shall also be used to obtain samples for index testing and determining engineering properties. In addition, drill holes shall be used to perform *in situ* tests and to install instrumentation.

8.1.1 Drilling Methods

Drilling shall generally be performed by the rotary diamond drilling method either with or without core recovery, as required. If core recovery is not desired, the rotary percussion method may be used for drilling in soil and rock.

8.1.2 Rotary Diamond Drilling

Core drillings shall generally be drilled vertically; however, where required, inclined and horizontally oriented drillings shall be performed to adequately define stratification, jointing and other discontinuities. Inclined drilling shall be used to investigate steeply inclined jointing in abutments and valley sections for dams, along spillway alignments and in foundations.

8.1.2.1 Drilling Equipment

Rotary diamond drilling shall be carried out using truck-mounted rotary drilling rigs. In areas with poor access, skid-mounted rigs may also be used.

Core Barrels

Cored rock samples shall be retrieved with hollow core barrels equipped with diamond- or carbide-embedded bits. For quick drilling and quality core retrieval, cores shall normally be drilled with a double to triple-tube core barrel with wire line system. A single-tube core barrel may be employed in hard, sound rock.

Core Size

The drill core size shall be decided based upon anticipated foundation conditions, laboratory testing requirements and the engineering information desired. Generally, the NW (54.7 mm diameter) or higher size cores shall be used. In soft, weak or fractured strata, larger diameter core barrels shall be preferred to ensure better recovery.

8.1.2.2 Drilling Procedure

For drilling, the casing shall be seated on bedrock or a firm formation to prevent traveling of the borehole and loss of drilling fluid. The surface of the rock or hard formation at the bottom of the casing shall be leveled, when necessary, using appropriate bits.

Core drilling shall be stopped on encountering soft materials that produce less than 50% recovery. It shall be resumed when hard formation is encountered again. If conditions prevent the continued advance of core drilling, the hole shall be cemented and re-drilled, reamed and cased, or cased and advanced with the next smaller-size core barrel.

The depth of drill holes shall be determined as per site geological conditions. Drilling shall be continued until core blockage occurs or until the net length of the core barrel has been drilled in.

8.1.2.3 Drilling Records and Observations

During drilling, procedural information on the drilling operation shall be recorded. Such records shall contain information on, but not limited to, the following:

- Details of bits and casings used, including size and type of core bit and barrel used, bit changes, size, type and depth of casing, casing shoe and/or casing bit used, etc.
- Problems or observations made during placement of the casing.
- Change in depth of casing setting during drilling.
- Depth, length and time for each run.
- Length/depth of pull (the actual interval of core recovered in the core run).
- Amount of core actually recovered.
- Amount of core loss or gain.
- Amount of core left in the hole (tape check).
- Depths at which in situ tests were done, etc.

In addition, the following observations shall be carefully recorded during the drilling:

a. Colour of return drill water and the depths at which its color changes.
b. Depth at which drill water is lost or is partially or fully returned.
c. Approximate water intake values in zones where total or partial water loss is observed.
d. Temperature of hot water, if any, including its depth of occurrence and amount of flow.
e. Rate of penetration, including depths where the rate shows marked changes.
f. Any sudden fall of the drill rod and the depth at which such fall occurs.
g. Depth at which a core loss might have taken place.
h. Blockage of drill bit during drilling and recovery of re-drilled and rounded core pieces.
i. Any heavy vibration or torque noted during drilling and their depth of occurrence.
j. Daily ground water level in drill holes, both before and after the drilling, and any fluctuations in levels observed therein.
k. Other peculiar conditions, including the following:
   - Depth at which drill hole is grouted.
   - Artesian conditions, including the depth, pressure and volume of outflow of water.
   - Gas discharge.
   - Deviation of inclined drill holes, if any, during drilling in each run.
   - Mineralization, if any, encountered, etc.

Suggested formats for recording drilling information and observations are included in Annexes 1D – A and 1D – B.

8.1.2.4 Core Logging

Cores shall be logged at the drilling site immediately after drilling. Colour photographic records of all cores shall also be made as soon as possible after their retrieval.

Core logs shall provide an accurate and comprehensive record of the geologic conditions encountered during drilling. They shall contain all the information obtainable from the core pertaining to the rock and discontinuities. The logs shall be graphical and/or tabular in nature and shall include, as a minimum, the following information:

a. Elevation with coordinates;
b. Description of lithology using standard symbols;
c. Systematic rock description for each lithologic unit, including unit designation, rock type, weathered state, texture, structure, colour, grain size, rock material strength, mineral rock type and rock name.
d. Size of core pieces;
e. Structural condition;
f. Discontinuity spacing;
g. Core recovery;
h. RQD;
i. Fracture frequency;
j. Size of hole;
k. Casing;
l. Depth of water level;
m. Drill water loss;
n. Permeability;
o. Penetration rate;
p. Special observations and interpretations.

A suggested format for core logging is included in Annex 1D – C.

8.1.2.5 Storage and Transportation

Recovered cores shall be stored in durable core boxes generally 1 m long with 5 rows. The project name, location and number of drill holes, box number and drilled depth shall be clearly marked on the boxes.

Soft or friable cores, or those which change materially upon drying, shall be wrapped in plastic film or sealed in wax. Any noticeable gaps in recovered cores, which might indicate a change or void in the formation, shall be indicated using properly marked spacer blocks or slugs. Fractured, bedded and/or jointed pieces of core shall be reassembled in the sequential order of their recovery before they are kept in the core box.

During transportation, core boxes shall be padded or suspended to isolate them from shock or impact caused by rough terrain or careless operation.

8.2 Test Pits and Trenches

Test pits and trenches shall be used for subsurface investigations at sites with limited depth of overburden or at sites where structures with shallow foundations are proposed. These methods shall also be employed for determining the geomorphic history of sites, collecting samples for laboratory tests and conducting *in situ* tests. Trenches may also be used in investigation of faults and weak zones in rock.

8.2.1 Excavation

Test pits and trenches shall be excavated manually or with mechanical equipment such as bulldozers, backhoes, draglines or ditching machines. If available, mechanical excavation shall be preferred for shallow pits and long trenches.

Dimensions

The depth of test pits and trenches shall depend on the requirements of the investigation. Generally, it shall be limited to a few meters below the ground water table.

Pits and trenches shall be dimensioned to ensure that their side slopes are stable and that sufficient working space is available at their bottom. Generally, a minimum dimension of 1.2 m in each direction shall be maintained at the bottom of the pits and trenches. If required, additional space for sheeting, supports, hoisting arrangements, ladder, etc. shall be provided at the bottom.

Trenches shall be excavated as a single slot trench or as a series of short trenches spaced at appropriate intervals. On slopes, excavation shall proceed from the top of the slope to its bottom.

Side Support

Unless determined to be inherently stable, all sides of pits and trenches shall be adequately supported with sheeting and bracing or cribbing with timber. Unless dug in hard soils, pits and trenches with depth exceeding 6 m shall not be left unsupported.

Dewatering

Excavation for test pits and trenches that extend below the water table shall proceed only with continuous pumping of water. For this purpose, a suitable dewatering system shall be designed and installed at the investigation site before excavation proceeds below the water table.

Ventilation

Deep test pits shall be ventilated to prevent accumulation of dead air. This shall be achieved through ventilation pipes extending from the floor of the pit to its mouth. If required, forced ventilation shall be resorted to.

8.2.2 Sampling

During excavation, samples of the excavated materials shall be taken for every change in strata. The samples shall be taken as soon as the materials are excavated, and their natural water content shall be determined immediately.

As far as practicable, undisturbed samples shall be obtained from each stratum exposed in the pits and trenches. If the size of the pit permits, a pillar of suitable dimensions shall be left undisturbed at the centre of the pit or trench to collect undisturbed samples of the required size. Special care shall he taken to preserve the natural moisture content of the samples.

8.2.3 Logging

Test pits and trenches shall be systematically logged. Where the strata are irregular in shape, three-dimensional logging may be carried out. These logs shall contain information on the following:

a. Soil or rock types penetrated and their nature and condition;
b. Position and attitude of contacts, faults, joints, shear zones and clay seams;
c. Elevation and fluctuations in ground water level, water inflows during excavation or any other information bearing on the ground water conditions, such as pumping;

All walls and base of the test pits and trenches shall be photographed. In addition, the following records shall be maintained:

- Records of percolation tests, if any;
- Record of samples for testing, including the location of their collection, date and methods of sampling, etc.

8.2.4 Documentation

The information collected from pits or trenches shall be properly documented in log sheets and through plans and sections. A suggested format for log sheets is shown in Annex 1D – B.

Geological logs of pits and trenches shall be drawn at a scale of 1:100. Likewise, plans and sections showing exploratory features shall generally be drawn at a 1:1,000 scale. In these documents, different formations and their physical conditions shall be shown by appropriate symbols or colors.

8.2.5 Safety Measures

While excavating or investigating pits and trenches, the following safety measures shall be adopted:

a. Deep test pits and trenches shall be ventilated to prevent accumulation of dead air. Special precautions shall be taken for this purpose if there is likelihood of the presence of obnoxious gases.
b. Test pits and trenches left open for inspection shall be provided with covers or barricades, and they shall be suitably fenced.
c. Pits and trenches shall be filled back properly when exploration and physical inspections are completed and relevant records have been obtained.

8.3 Ground Water Studies

Ground water studies shall be performed in the headworks area to generate data for design of dewatering systems required during headworks construction. They shall also be required to study the potential for interference to aquifers by the construction of the headworks.

Depending on the size and nature of the headworks, ground water studies may range from broad regional studies at headworks of pondage run-of-river schemes to site-specific studies such as pumping tests for relief well design. In view of their complexity, these studies shall be undertaken only after a thorough analysis of existing and easily available data.

8.3.1 Procedures

Ground water studies shall consist of observation and measurement of ground water levels and flows in the headworks area. This shall be achieved through a study of existing wells, boreholes, observation wells, piezometers and springs in the area. Geophysical methods and tracer testing may also be adopted for this purpose.

8.3.1.1 Wells

Ground water levels in wells shall be obtained from sounding or from records maintained by well owners. Where available, information on the following issues shall also be collected:

- Pumping quantities;
- Seasonal variations in ground water and pumping levels;
- Depths of wells and screen elevations;
- Corrosion problems
- Settlement records attributable to ground water lowering from pumping.

8.3.1.2 Borings

Ground water levels shall be obtained from drilling logs. While doing so, the influence of soil types, drilling fluids and time of reading after initial drilling on the water level readings shall be borne in mind.

8.3.1.3 Piezometers and Observation Wells

Piezometers or observation wells shall be installed to determine ground water levels in the headworks area. They shall also be used to collect data on the fluctuations in ground water levels over time through manual or automated periodic readings.

The locations of piezometers or observation wells shall be selected based on information available on the regional ground water regime. Where acceptable, they may also be installed in existing boreholes.

All piezometer borings shall be carefully logged. Conditions encountered in the borehole related to perched or confined aquifers shall be carefully evaluated while selecting screened intervals for piezometers and interpreting data obtained from them.

8.3.1.4 Springs and Surface Water

Springs in the headworks area shall be monitored to obtain their water elevations and flow rate. The flow rates shall be measured during dry and wet seasons to determine the influence of rainfall on seepage conditions. Soil or rock strata at the springs shall be evaluated to locate permeable horizons. In addition, the elevation of water levels in lakes and ponds shall be measured during wet and dry seasons to evaluate the extent of surface water fluctuations.

8.3.1.5 Geophysical Methods

Geophysical methods shall be used to characterize the hydrogeology of a site. Among the various geophysical methods, the seismic refraction method, discussed in Section 7.3.2, may be used to determine the depth to saturated material. Likewise, surface resistivity surveys, discussed in Section 7.3.3, may be employed to assess the presence of and depth to water. The geophysical data thus obtained shall be verified from piezometers installed in the study area.

8.3.1.6 Tracer Tests

Tracer tests may be conducted at headworks sites with karst terrains to assess the direction and rate of flow in the ground water system in the area. In these tests, a tracer element, which could either be environmentally benign dyes or biological tracers such as pollen, shall be introduced into a boring, observation wells, piezometers or other access points. The tracer element shall be monitored at an exit point such as a spring, and the travel time from the introduction to the detection shall be recorded. By running numerous tests at different locations, a picture of the ground water flow regime shall thus be developed.

8.3.2 Interpretation of Data

Data from ground water studies shall to develop a complete hydrogeological picture of the headworks area. This shall be achieved by interpreting the data in conjunction with available site and regional geologic information to determine the following information related to the ground water in the headworks area:

a. Water table or piezometric surface elevations and profiles;
b. Fluctuations in water table elevations;
c. Possible existence and location of perched water tables;
d. Depths to water-bearing horizons;
e. Direction and rate of seepage flow.

8.4 Field Permeability Tests

Field permeability tests shall be conducted to determine the permeability of each subsurface stratum encountered in the overburden and in the bedrock present in the headworks area. This shall be done using one of the following field methods:

a. Constant head method;
b. Falling head method;
c. Pressure tests.

Of these methods, the constant and falling head methods shall be used for determining the permeability of overburden. Pressure tests shall be conducted to estimate the permeability of bedrock.

8.4.1 Constant Head Method

The constant head method shall consist of computing the permeability of a stratum from data on steady state constant discharge in a test hole measured with the water level in it kept constant. This method shall be used when the stratum being tested is expected to have high permeability, e.g. a stratum with free draining material such as granular soil or heavily jointed rock mass.

8.4.1.1 Procedure

The constant head test shall be carried out in boreholes drilled for subsurface explorations or for installing piezometers. The borehole shall be drilled to the level at which the test is to be performed, and a casing shall be driven in simultaneously. After the desired level for the test is reached, the hole shall be cleaned, and the test shall be conducted by allowing clean water to flow in the hole under gravity at constant head and measuring the consumption of water per unit time.

8.4.1.2 Observations

The observations of the test shall be suitably recorded in appropriate format.

8.4.1.3 Computation of Coefficient of Permeability

Using the results of the constant head method, the coefficient of permeability, K, shall be obtained using the following relation (IS: 5529 Part 1 – 1995):

Eq. 4
$$K = \frac{Q}{5.5rH}$$

where Q is the constant rate of flow into the hole, r is the internal radius of the casing and H is the differential head of water.

8.4.2 Falling Head Method

The falling head method shall consist of computing the equivalent permeability of a stratum from data on the rate of fall of water level in a test hole. It shall be used for strata with low permeability, e.g. strata with slow draining material such as jointed rock mass or fine grained soil like fine sand, silt or clay.

8.4.2.1 Procedure

In this method, boreholes for subsurface explorations or for installing piezometers shall be drilled up to the bottom of the test horizon and cleaned. Thereafter, a packer shall be fixed at the desired depth such that it enables testing of the full section of the hole below itself. The test shall be conducted by filling up a stand pipe to its top with water and measuring the rate of fall of water inside the pipe.

8.4.2.2 Observations

The observations of the test shall be suitably recorded in appropriate format.

8.4.2.3 Computation of Coefficient of Permeability

For an uncased hole, the coefficient of permeability, K, shall be obtained from the results of the falling head method using the following relation (IS: 5529 Part 1 – 1995):

Eq. 5
$$K = \frac{d^2}{8L}\log_e \frac{L}{R}\frac{\log_e h_1 / h_2}{t_2 - t_1}$$

where d is the diameter of the stand pipe, L is the length of test zone, h_1 and h_2 are the heads of water in the stand pipe at times t_1 and t_2, respectively, above the piezometric surface and R is the radius of the hole.

8.4.3 Pressure Test

Pressure tests shall be performed to measure the permeability of strata within the bedrock. Its results shall also be used to assess leakage in the foundation, to estimate the extent and pattern of grouting required and to assess the groutability of the bedrock zones.

Pressure tests shall typically be conducted during exploratory core drilling. They shall form an integral part of exploratory core drilling in all cases where rock seepage characteristics could affect project safety or economy.

8.4.3.1 Methods of Testing

Pressure test shall be carried out in drill holes using one of the following methods:

a. Single packer method;
b. Double packer method.

The single packer method shall be used where permeability values are required side by side with drilling, e.g. at sites where multiple aquifers are present, or where the full length of the hole cannot stand without casing or grouting, e.g. in soft rocks such as sand rock, clay shale, etc. or in highly fractured or sheared rocks. The double packer method shall be used where the rock is sound and the full length of the hole can stand without casing or grouting. Wherever time permits, the single packer method shall be preferred.

8.4.3.2 Test Intervals

Pressure tests shall typically be conducted at intervals of 1.5 to 3 m, but the interval may be varied to fit specific geological conditions observed during core drilling operations. Such conditions shall be determined by examining freshly extracted cores, noting depths where drilling water was lost or gained, noting drill rod drop and conducting downhole geophysical surveys.

8.4.3.3 Test Sections

Irrespective of the method used, the test section for pressure testing shall not be less than five times the diameter of the borehole. This section shall be confined by means of a packer or packers, depending on whether the single or the double packer method is used.

8.4.3.4 Test Pressures

Pressure tests shall be conducted by pumping water into the test section under different pressures. Each pressure shall be maintained until the water intake under that pressure becomes constant. The pressures shall start from a low value and be increased up to the limiting pressure causing uplift or the sum of the hydraulic head to which the stratum would be subjected by the proposed structure and losses due to friction, whichever is less. In order to prevent upheaval of rock foundations, the test pressures for permeability tests shall generally be limited to those listed in Table 3. Higher pressures may be needed for assessing groutability.

Table 3: Limiting test pressures for permeability tests in test sections

Rock type	Pressure per meter of rock load (MPa)
Unconsolidated or poorly consolidated sedimentary formations	0.012
Consolidated horizontally bedded sedimentary formations	0.018
Hard igneous and metamorphic rocks	0.024

(Source: IS: 5529 Part 2 – 1985)

8.4.3.5 Data Recording

During pressure tests, the injection pressures and the corresponding rate of water intake shall be noted and presented as suggested in Annex 1D – C.

8.4.3.6 Interpretation

Using the data obtained from pressure tests, curves of water intake versus pressure shall be plotted to analyze specific foundation problems. The permeability values shall be indicated in terms the lugeon value, L_u, calculated using following equation:

Eq. 6

$$L_u = \frac{10Q}{PL}$$

where Q is the quantity of water injected in liters per minute, P is the injection pressure in kg/cm^2 and L is the length of the test section in m.

Qualitative evaluation of leakage and grout requirements shall be made from raw pressure test data. Calculations to estimate seepage quantities shall be performed by converting pressure test data into values of equivalent permeability or transmissivity.

8.5 *In Situ* Testing

In situ tests on subsurface soil and rock shall be performed to determine their engineering properties. Depending on the material occurring in the foundations and abutments of the headworks structures, these properties shall include compressive strength, shear strength, *in situ* state of stresses, deformation properties, dynamic moduli, permeability and liquefaction potential. A list of recommended tests for determining engineering properties of soils and rocks is presented in Table 4.

8.6 Backfilling Boreholes and Exploratory Excavations

Except where they are required to be preserved for future use, all boreholes, test pits and trenches shall be backfilled after completion of investigations. As a minimum, borings that need to be preserved for future use shall be protected with a short section of surface casing, capped and identified. Test pits and trenches shall be provided with suitable covers or barricades until they are backfilled.

8.7 Disposition of Samples

Soil and rock samples obtained during subsurface investigations may be discarded after the tests intended on them are over. Rock cores not used for tests shall be properly preserved, boxed and stored in a protected facility until permanently records of detailed logs, photographs and test data have been made. However, a few cores related to future construction and a few selected cores representative of foundation and abutment conditions shall be retained. Selected cores retained after the completion of the headworks may be discarded or otherwise disposed off five years after final completion of the headworks, provided no unforeseen foundation or abutment conditions have developed.

9. LARGE-SCALE, PROTOTYPE TESTS

Large-scale prototype tests shall be carried out at large or complex headworks to confirm design assumptions, to improve the confidence level in design and reduce safety factors and to verify constructability of the proposed design. These tests shall consist of the following:

a. Exploratory adits;
b. Shafts;
c. Grouting tests.

In view of the time and cost required to carry them out, these tests shall be utilized on major projects and when no other method provides the required information.

9.1 Exploratory Adits

Exploratory adits shall be excavated to conduct detailed examination of the continuity, nature and structure of particular geological formations, including joints, fractures, faults, shear zones, other discontinuities and solution channels. They shall also be used to explore conditions at the locations of the foundations and abutments for large dams and to establish the minimum excavation limits to reach fresh and sound rock. They shall serve as a source for collecting undisturbed samples for laboratory testing and as a means for conducting i*n situ* tests, such as those for *in situ* state of stress, strength parameters and related data, at locations and in directions corresponding to the structural loading. If required by the site geological conditions, suitable instrumentation, such as convergence measuring points, extensometer, load cells and pore pressure measuring devices, may also be installed and monitored in the adit.

Table 4: In situ tests for soils and rocks

Purpose of Test	Type of Test	Applicability to	
		Soil	Rock
Compressive strength	Pocket penetrometer test	√	
	Schmidt rebound test		√
Shear strength	Standard penetration test	√	
	Becker penetration test	√	
	Direct shear test	√	√
	Field vane shear test	√	
	Plate bearing or jacking test	√	√[1]
	Cone penetrometer test	√	
	Borehole direct shear test[2]	√	
	Pressuremeter test[2]		√
	Uniaxial compressive test[2]		√
	Borehole jacking test[2]		√
Bearing capacity	Standard penetration test	√	
	Plate bearing test	√	√
Stress conditions	Hydrofracture Method	√	√
	Pressuremeter Test	√	√[1]
	Overcoring Method		√
	Flat jack		√
	Uniaxial (tunnel) jacking	√	√
	Borehole jacking[2]		√
	Chamber (gallery) pressure[2]		√
Mass deformability	Geophysical (refraction) test	√	√
	Pressure meter or dilatometer	√	√[1]
	Plate bearing test	√	√
	Standard penetration test	√	
	Uniaxial (tunnel) jacking test	√	√
	Borehole jacking[2] test		√
	Chamber (gallery) pressure[2]		√
Relative density	Standard penetration	√	
	In situ sampling	√	
	Cone[2] penetrometer	√	
Liquefaction potential	Standard penetration test	√	
	Cone penetration test[2]	√	

[1]Primarily for clay shales, badly decomposed or moderately soft rocks and rock with soft seams.

[2]Less frequently used

(Source: USACE EM 1110-1-1804: Geotechnical Investigations)

9.1.1 Excavation

Exploratory adits shall generally be excavated using drill and blast techniques. They shall be provided with a low outward floor slope to make them self-draining.

Dimensions

The dimensions of the adit shall depend on the type of rock mechanic tests to be conducted in it and the space requirements of the testing machine equipment. Generally, a minimum clear width of 1.5 m and height of 2 m shall be provided. A rectangular section may be adopted in hard rocks while an arched roof may be used in soft rocks.

Support

Depending upon site conditions, the roof and the sides of the adit shall be supported with timber, rock bolt or steel sets. The size and spacing of the supports shall be decided based on the character of the formation and the spacing and attitude of joints therein.

Ventilation and Lighting

The entire length of the adit shall be properly ventilated to remove pockets of foul air or blast gases. It shall also have adequate lighting arrangements for proper examination and recording of data.

9.1.2 Mapping and Sampling

Wherever possible, mapping and sampling of the exploratory adit shall proceed concurrently with its excavation. The adit shall be properly cleaned before these activities commence.

Detailed geological mapping of the adit crown, walls and invert shall be performed at a scale ranging between 1:50 to 1:200. The mapping shall include details of geological materials and features listed in Section 7.1.1.2 for areal mapping. For complex geological environments and major projects, a three-dimensional map may be plotted to show the actual disposition of geological features.

9.1.3 Documentation

The information collected from exploratory adits shall be properly documented in log sheets at a scale of 1:100. Plans and sections showing exploratory features shall generally be drawn at a 1:1,000 scale.

9.2 Shafts

Shafts shall normally be employed to reach either a particular formation at a great depth (exceeding about 6 m) or to extend the exploration below river bed by means of tunnels.

9.2.1 Excavation

Shafts shall be dug manually or with mechanical equipment. In manually dug shafts, the materials shall be removed by buckets operated by hoists or windlass equipped with a ratchet device for safety.

Dimensions

Shafts may be rectangular or circular in section and shall have adequate space for movement of men, equipment and other accessories. Rectangular sections shall have a minimum dimension of 2.4 x 2.4 m while circular sections shall have a minimum diameter of 2.4 m.

Side Support

In weak and caving ground, the sides of shafts shall be supported with sheeting and bracing or cribbing with timber. The spacing and size of the supports shall depend on the nature of the strata.

Dewatering

If water is encountered, shafts shall be dewatered continuously to enable further progress. For this purpose, a suitable dewatering system, preferably using electrical pumps, shall be designed and installed at the investigation site.

Ventilation

Deep shafts shall be ventilated to prevent accumulation of dead air or blast gases. For this purpose, ventilation pipes from the floor of the shaft to its mouth shall be used. Air from a compressor or blower may also be used.

9.2.2 Mapping

Shafts shall be mapped in a manner identical to exploratory adits. For minor projects and geologically simple formations, a 1 m strip log of the shaft wall may be adequate. For major projects and in geologically complicated formations, a three-dimensional log showing the actual disposition of the geological features may be required.

9.2.3 Documentation

The information collected from shafts shall be properly documented in log sheets at a scale of 1:100. Plans and sections showing exploratory features shall generally be drawn at a scale of 1:1,000.

9.3 Test Grouting

Test grouting shall be performed at headworks sites where complex geological conditions to acquire knowledge of the extent to which the subsurface materials are groutable. It shall consist of experimental grouting operations on exploratory boreholes. Information obtained from this test shall be used to formulate grouting procedures and to determine design specifications, costs and appropriate equipment.

10. LABORATORY INVESTIGATIONS

Laboratory tests shall be conducted to complement *in situ* tests in investigating physical properties of soils and rocks and in selecting geotechnical design parameters for headworks structures. These shall consist of the following types of tests on soils and rocks:

a. Index and classification tests;

b. Tests for engineering properties.

10.1 Sampling Locations

Sampling of soils and rocks for index and engineering property tests shall be planned to ensure adequate representation of stratigraphic units in the vertical and lateral directions. The sampling shall be statistically balanced and shall ensure that coverage of field conditions is regular.

10.2 Tests for Soils

The laboratory tests typically required for classifying soils and determining their index values and engineering properties are listed in Table 5.

10.3 Tests for Rocks

Laboratory tests typically required for classifying rocks and determining their index values and engineering properties are listed in Table 6. Among the index and classification tests, water content, unit weight, total porosity and unconfined compression tests shall be performed on representative cores from each major lithological unit to characterize the range of properties in the headworks area. Petrographic examination and additional tests for specific gravity, apparent specific gravity, absorption, elastic constants, pulse velocity and permeability shall be carried out depending on the nature of samples or the project needs.

Table 5: Laboratory tests for soils

Purpose of test	Type of test
Index and classification	Soil classification
	Water content test
	Liquid and plasticity limit tests
	Grain size distribution test
	Relative density test
	Slaking test
	X-ray diffraction test
Engineering properties	Unconfined compression test
	Direct shear test
	Permeability test
	Consolidation test
	Triaxial shear test

(Source: USACE EM 1110-1-1804: Geotechnical Investigations)

Table 6: Laboratory tests for rocks

Purpose of test	Type of test
Index and classification	Water content test
	Specific gravity test
	Sonic velocity test
	Rebound number test
	Permeability tests
	Petrographic examination
	Specific gravity and adsorption test
	Unit weight and total porosity test
	Durability test
	Los Angeles abrasion test
	Point load test
	Unconfined compression test
Engineering properties	Unconfined compression test
	Triaxial compression test
	Direct shear test
	Triaxial shear test
	Brazilian test
	Tensile test

(Source: USACE EM 1110-1-1804: Geotechnical Investigations)

11. CONSTRUCTION MATERIAL INVESTIGATION

Construction material investigations shall examine the availability of the requisite quality and quantity of construction materials at borrow areas and quarry sites identified from desk studies or field visits. The investigations shall focus on the following materials:

a. Concrete aggregates;

b. Embankment soils;

c. Riprap and rockfill.

Of these, concrete aggregates shall be needed for reinforced cement concrete structures and plain cement concrete used in the headworks. Soils and rockfill shall be required for the various zones of embankments. Likewise, riprap shall for used for protecting embankment slopes and other surface slopes in the headworks area.

11.1 Investigations for Concrete Aggregates

Investigations for concrete aggregates shall be carried out to select suitable sources for fine and coarse aggregates. The investigations shall consist of prospecting for aggregate sources, geological mapping, subsurface explorations and laboratory testing.

11.1.1 Prospecting

Prospecting for concrete aggregates shall be aimed at identifying potential sources of coarse and fine aggregates in the vicinity of the headworks. Such sources shall include deposits of natural aggregates, talus or quarries in areas of bedrock outcropping.

The prospecting shall generally commence from river deposits, debris flow or alluvial fans upstream and downstream of the headworks. At high elevations, glacial deposits influenced by fluvial agencies shall also be searched. Where suitable or adequate natural aggregates are not available, the prospecting shall extend to rock deposits from which aggregates can be produced through quarrying and processing of rocks.

11.1.2 Site Mapping

Each identified source of aggregates shall be mapped on available topographical maps of the concerned area. For mapping natural deposits, the area shall be traversed and information on the following features of the deposits shall be collected through visual inspection and appropriate topographical and geological techniques:

a. Size, shape, location and thickness of deposits;
b. Geologic character of deposits, including their genetic origin;
c. Grading, rounding and degree of uniformity of the aggregate particles.

Likewise, mapping for rock quarries shall lead to collection of information on the following:

a. Types of rocks and their properties;
b. Condition of rocks, including their degree of weathering;
c. Bedding and foliation planes;
d. Prominent and random joint sets, including their persistency;
e. Presence of layers or zones of undesirable materials such as clay or shale;
f. Ground water level.

Based on the information collected, borrow or quarry source map shall be produced. These maps shall show the boundaries of the natural deposits and rock quarries, their estimated thicknesses, soil types and rock types with adequate descriptions of surficial weathering, hardness and joint spacing.

During preliminary investigations, source maps shall be prepared at a scale of 1:25,000 or 1:50,000. However, larger scale maps, usually 1:10,000 or 1:15,000, shall be prepared during detailed investigations.

11.1.3 Subsurface Investigations

Subsurface investigations shall be conducted at the most promising deposits to estimate the quantity of construction materials available and to evaluate their engineering properties. The investigations shall be made by means of cased test holes, open test pits or trenches using methods discussed in Section 8.2.

Disturbed samples of concrete aggregate materials shall be obtained from test pits, trenches and cased auger bore holes. Since the gradation of concrete aggregates is of great value, portable screening apparatus may sometimes used to determine the grading of the samples in the field. Representative samples of aggregate shall be subjected to laboratory tests to determine the physical and chemical properties of the material.

11.1.4 Laboratory Tests

Laboratory tests on concrete aggregates shall consist of the following:

- Initial laboratory acceptance tests for suitability, concerned with the different properties.
- Secondary labotratory tests on approved samples to determine physical properties used in mix design, such as sieve analysis, absorption, bulk density, compacted unit mass, voids and bulking.
- Field tests for local acceptance or control, such as sieve analysis, deterious materials, moisture content and compacted unit mass.

The realibility of the tests shall be ascertained by the repeatability (testing samples repeatedly) and reproducibility (testing by reducing the sample size) of the test. Laboratory tests shall include durability, mechanical, physical and chemical test.

11.1.4.1 Suitability

The suitability of a material for use as aggregate is tested by durability test. For suitability test, study of grain size, specific gravity, moisture content, durability, compressive strength, density and shear strength is checked.

11.1.4.2 Durability Test

The durability test shall measures resistance to mechanical abrasion (los angeles abrasion value, and the polish stone value), break-down under repeated cycles of wetting and drying using water and other solvents (slake durability index, sulphate soundness test and other physico-chemical tests), and the content of organics, chlorides and sulphates by chemical tests. The reaction between rock fragments and cement shall be measured by alkali-reactivity tests. These tests shall consist of alkali-silica, alkali-silicate and alkali-carbonate reactions.

11.1.4.3 Mechanical, Physical and Chemical Tests

The mechanical, physical and chemical tests listed in Table 7 may be conducted on concrete aggregate.

11.2 Investigations for Embankment Soils

Investigations for embankment soils shall be conducted to select suitable sources for the following materials for embankments:

a. Permeable material (rock material).
b. Semi-permeable materials for filters and transitions.
c. Earth materials for impervious cores.

The investigations shall consist of prospecting, geological mapping, subsurface explorations and laboratory testing.

11.2.1 Prospecting

Investigations for permeable material shall aim at identifying coarse sand, gravel deposits containing stones of different size in riverbed or river terrace, crushed rock and excavated material from foundation for use as filter.

Table 7: Mechanical, physical and chemical tests for concrete aggregates

Mechanical tests	• Aggregate crushing value test • Aggregate impact test • Point load test • Schmidt rebound test • Roughness test • Triaxial compression test • Shear test • Deformation test • Sonic velocity test etc.
Physical tests	• Sieve analysis • Shape analysis • Density measurement • Water absorption • Permeability measurement • Study of surface texture • Petrographic examination • Flotation test
Chemical tests	• Chloride content test • Sulphate content test • Organic content test • Methylene blue absorption test • Aggregate bitumen affinity test • Acid solubility test • Alkali reaction test

Investigation for semi-permeable materials shall be performed to borrow-area capable of supplying the material which is hard, has required shear strength, contains no harmful substance such as organic matters, and has proper permeability and grain size distribution appropriate to the purpose of transition of water cut-off layer and permeable layer. The investigation shall focus on fine grained sand and gravel obtained from riverbed and river terrace, fine grain crushed rock and material excavated from foundation excavation.

Investigations for core material shall aim at identifying borrow area having required coefficient of permeability and shear strength, small amount of compression, easiness for compaction and no organic substances.

The prospecting shall generally commence from the areas of structural excavation for the headworks structures, especially if a considerable amount of excavation is required to reach a competent foundation level. If sufficient embankment material is not available from the excavations, all potential areas within a radius of two kilometers of the dam site shall be investigated before more distant sources are considered.

11.3 Investigations for Rockfill and Riprap

11.3.1 Prospecting

Prospecting for riprap, rockfill and aggregates shall start from the vicinity of the headworks and continue outwards radially until sources which are suitable in quality and sufficient in volume to fulfill the anticipated construction requirements are located. Generally, the range of investigations shall be limited economically to about 20 km from the headworks site for sand and gravel and to between 10 to 15 km for rocks, and the nearest borrow area shall be selected.

The prime criteria for rock sources as riprap are quality and size of the rock fragments. Those who perform the investigations shall attempt, by inspection, to evaluate the ability of the rock to resist wave action, freezing and thawing and other disintegrating forces and to determine whether the deposit will yield sufficient material for the required sizes.

11.3.2 Subsurface Investigations

Riprap samples shall be obtained by blasting down an open face on the sidewall of a test pit, trench or exposed ledge to obtain unweathered fragments representing each type of material as it will be quarried and used in riprap. Intervening layers of soil, shale, or other soft rock obviously unsuitable for riprap need not be sampled, but full description of these materials shall appear in log charts and in the report of investigation.

12. DOCUMENTATION

Upon completion of the geological and geotechnical investigations, the geological and geotechnical investigative data, including results of laboratory and field tests, shall be documented. The documentation shall present a comprehensive assessment and description of the geology of the project. It shall be limited, however, to an effective combination of brief discussions, tabulated data, and geological illustrations to depict the conditions that are of engineering significance. The documentation shall focus on the following topics:

a. Significant and controlling topographic conditions.
b. Description of all aspects of bedrock and recent geology, including discussions of:
   i. Composition and structure of the rock.
   ii. Engineering description of soils and of their relationship to the bedrock.
   iii. Engineering properties of rocks and soils determined by investigations.
   iv. Geologic conditions that present special engineering problems.
   v. Remedies proposed or used for the special problems.
   vi. Sources and characteristics of construction materials.

The surface and subsurface investigations, laboratory tests and geological illustrations in geotechnical reports shall be sufficiently comprehensive to supply reliable information on all geological conditions that can influence the design, construction and cost of the project. The following maps and test data shall be included in the documentation as applicable.

- Headworks location map.
- Regional geological map.
- Headworks area geological map or engineering geological map.
- Reservoir geological map.
- Plan of geological exploration.
- Geological logs of drilling work.
- Plan of geophysical exploration.
- Geological section by geophysical exploration.
- Laboratory test data and plots.
- Rock counter map for important structures.
- Geological sections along important civil structures.
- Map showing spoil banks.
- Foundation treatment map.
- Slope support map.
- Construction material quality and quantity.

13. PROTECTION OF ENVIRONMENT

While conducting site geological and geotechnical investigations, necessary measures shall be adopted to protect the environment of the headworks areas and its surroundings. During the investigations, the following protective measures shall be enforced:

- Access routes to the investigation sites shall be selected with care to minimize damage to the environment.
- Operation of investigation equipment shall be controlled at all times, and the extent of damaged areas shall be held to the minimum consistent with the requirements for obtaining adequate data.
- Sediment flow from the investigation sites into water bodies shall be kept within permissible levels specified in environmental regulations.

After the investigations are complete, areas disturbed by the investigations shall be restored to a natural appearance.

Annex 1D – A.1: Daily Drill Report

Project			**Location**		**Feature**	
Drill No. and type:					Pump No.:	
Screw feed/hydraulic feed:					Capacity and pressure used:	
Collar elevation:					Hole No.	
Ground elevation:					Co-ordinates:	
Date:					Bearing of hole:	
Shift: From… to …h:					Angle with vertical:	
Depth of water level:					Depth drilled during the shift:	
					From	To
At start of shift:						
At end of shift:						

Run		**Length Drilled mm**	**Type and Size of Hole**	**Colour of Return Water**	**Type of Soil / Rock**	**Water Loss with Depth**	**Core Recovery**			**Rate of Penetration mm/min**	**Remarks**
From m	To m						Length	Percentage	SI No. of Cores		
1	2	3	4	5	6	7	8	9	10	11	12

(Source: IS: 4464-1985)

Drill Foreman/Supervisor Operator

Team Leader Drill Observer

Drill Observer's Remarks

1. Water loss during drilling may either be recorded as: (i) Complete when no water is coming out; partial; or nil water loss; or (ii) In percent of return water [100% loss when no water is coming back and no water loss (0 percent) when all the drilling water is coming back].
2. Penetration speed in special zones (soft or broken zones); and other details of drilling like heavy vibration recorded during drilling.
3. Reasons for heavy core loss as integrated with speed of drilling
4. Any special conditions not recorded; for example, depth at which blasting was done while driving casing, depth at which hole was grouted, artesian water condition (if any observed) during drilling.
5. If water flows are encountered at the collar of the drill, then the pressure head and discharge at the collar should be recorded. On completion of the hole, the pressure decline over a period of time should also be recorded.

Annex 1D – A.2: CONSOLIDATED DRILLING LOG

Top R.L.		
Project Site:	Angle from vertical:	Machine:
Hole No.:	Bearing of angle hole:	Driller:
Location:	Water table R.L.:	Logged by:
Co-ordinate:	River water R.L.:	

												Percolation Test								**Penetration Test**		
														Water meter reading								
Date and time	**Size of hole**	**Depth of casing**	**Depth of hole**	**Length of drilled**	**Core run**	**Percent core recovery and RQD**	**Serial No. of cores**	**Wash water colour**	**Type of core barrel and bit used**	**Description of strata**	**Other drilling notes**	**Depth of slab**	**Pressure in N/mm2**	**Initial**	**Final**	**Loss in litres**	**Time in minutes**	**Loss in litres per minute**	**Lugeons**	**No. of blows**	**Penetration**	**Standard Promotion Test (S.P.T)Value**
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23

(Source: IS: 4464-1985)

Prepared by: Approved by:

Annex 1D – A.3: PROFORMA FOR PRESENTING DRILLING INFORMATION

Project:		**Hole No.:**
		Sheet No.:
Geological log of drill hole:		
Location:	Co-ordinates:	Feature:
Bearing hole:	Angle with horizontal:	Total depth:
Collar elevation:	Ground elevation:	Type(s) of core barrel:
Started:	Completed:	Drilled agency:

	Lithology		Size of core piece					Structural conditions		Percent core recovery												Drill Water loss		Permeability			
Elevation, m	Description	LOG	'10 mm	10 to 25 mm	25 to 75 mm	75 to 150 mm	∃ 150 mm	Description	LOG	0	20	40	60	80	100	Type of bit	R. Q. D	Fract Frequency/m	Size of hole	Casing	Depth of water level	Partial	Complete	Test section	mm/s or lugeon	Penetration rate mm/min	Special observations and interpretations

Organisation	Logged by	Approved by	Drawing No.
Department	Checked by		Date:
Government of			

(Source: IS: 4464-1985)

Annex 1D – B: LOG OF TEST PIT FOR BORROW AND FOUNDATION INVESTIGATIONS

Feature:	**Project:**	**Test Pit No.:**	
Area designation	Co-ordinates	Ground elevation	Depth of ground water level*
Method of excavation	Approximate dimension of hole	Date of excavation	Hole logged by

Classification symbol Letter /Graphic	**Depth (m)**	**Size and Type of sample taken**	**ɦ Classification & description of material (Geological and In-place Description for foundation**	**Volume of hole sampled (m^3)**	**Weight of 0 to 75 mm Material in Sample (kg)**	**Percentage by volume of 0 to 75 mm Materialɦ**	**Weight of 75 to 125 mm Material in Sample (kg)**	**Percentage by volume of 75 to 125 mm Materialɦ**	**Weight of Plus 125 mm Material in Sample (kg)**	**Percentage by volume of Plus 125 mm Materialɦ**	**Location of in-situ Tests**

Remarksɦ

(Source: IS: 4453 – 1980)

* Record after water has reached its natural level

ɦ Classification and identification of soils for general engineering purposes

ɦ (Weight of rock sampled x 100)/[(Bulk specified gravity of rocks) x (volume of hole sampled)]

ɦ Record water test and density test data, if applicable, and also bulk specific gravity stating how obtained (measured or estimated) under remarks.

Logged by: Approved by:

Annex 1D – C: PROJECT FORM FOR PRESENTATION OF DATA OF IN-SITU PERMEABILITY TESTS IN BEDROCK (IS CODE)

Project	**Date**
Drill hole No.	Diameter of drill hole
Feature	Total depth of the hole
Collar elevation	Depth tested: from to total
Height of the water swivel above the collar of the hole	Depth of ground water *
Type of drilling	Size of the drill rods/pipe
Note (if any)	

Test Section		**Meter reading of Water Intake Liters**				**Water Intake Liters/Minute**				**Water Pressure at collar MPa**	**Permeability in mm/s**
From	To	Initial reading	Reading after 5 minute	Reading after 10 minute	Reading after 15 minute	First 5 minute (col 4-3)	Second 5 minute (col 5-4)	Third 5 minute (col 6-5)	Average of Last Two Readingsɦ		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)

(Source: IS: 5529 (Part2) – 1985)

* Where permeability of several confined aquifers is tested the piezometric head of each aquifer should be recorded separately.

ɦ Test to continue till 3 consecutive readings of col. 10 are constant.

Test by: Approved by:

PART 1E – SEISMOLOGICAL STUDIES

TABLE OF CONTENTS

Part

1E

Seismological Studies

1. PURPOSE

Part 1E of the *Design Guidelines for Headworks of Hydropower Projects* establishes procedural guidelines for seismological investigations performed in support of design of headworks of run-of-river hydropower projects. The guidelines are intended to ensure adoption of uniform and standardized procedures in such investigations for headworks of hydropower projects.

2. SCOPE OF GUIDELINES

The guidelines presented in this document covers diversion structures generally present in hydroelectric development in Nepal. The guidelines are developed in accordance with standards and guidelines prevailing in different countries with due consideration to Nepali conditions and experience. These guidelines shall be basis of selecting seismic design parameters for design of diversion structures.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Acceleration time history	Time series of accelerations that is either recorded during an actual earthquake or synthesized to be representative of that recorded during an earthquake.
Active fault	A fault, reasonably identified and located, known to have produced historical earthquakes and which, because of its present tectonic setting, can undergo movement during the anticipated life of man-made structures.
Critical damping	Least amount of damping that will prevent free oscillatory vibration in an one-degree-of-freedom system.

Design response spectrum	Smooth response spectrum developed from a deterministic or probabilistic ground motion analysis and used as a basis for seismic design.
Earthquake	Sudden motion or vibration in the earth caused by the abrupt release of energy in the earth's lithosphere.
Earthquake magnitude	Measure of earthquake size, determined by taking the common logarithm (base 10) of the largest ground motion recorded during the arrival of a seismic wave type and applying a standard correction for distance to the epicenter.
Epicenter	The point on the surface of the earth directly above the focus (hypocenter) of an earthquake.
Fault	Fracture or zone of fractures in rock along which the two sides have been displaced relative to each other parallel to the fracture.
Fault plane	Plane that most closely coincides with the rupture surface of a fault.
Focal depth	Vertical distance between the epicenter and the hypocenter.
Hypocenter	Point within the earth at which an earthquake initiates.
Maximum Credible Earthquake (MCE)	Greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence.
Maximum Design Earthquake (MDE)	Maximum level of ground motion for which a structure is designed or evaluated.
Recurrence interval	Time between earthquakes or faulting events with specific characteristics in a specified region or in a specified fault zone.
Return period	Time between occurrences of ground motion with specific characteristics at a site.
Operating Basis Earthquake (OBE)	An earthquake that can reasonably be expected to occur within the service life of the project, i.e., with a 50-percent probability of exceedence during the service life.
Response spectrum	Plot of the maximum values of acceleration, velocity, and/or displacement response of an infinite series of single-degree-of-freedom systems subject to a time-dependent dynamic excitation.
Seismic hazard	Any physical phenomenon associated with an earthquake that may produce adverse effects on human activities.
Seismicity	Occurrence of earthquakes in space and time.
Seismic wave	An elastic wave in the earth usually generated by an earthquake source or explosion.
Strain (Elastic)	Geometrical deformation or change in shape of a body.
Strong ground motion	The shaking of the ground made up of large-amplitude seismic waves of various types.
Tectonics	Study of the earth's large-scale structural features

4. OBJECTIVES OF INVESTIGATION

Geological and seismological investigations of diversion structure sites shall be performed for projects located in seismic zones to derive meaningful seismic parameters necessary to ensure the earthquake safety of headworks structures based on requirements mandated by the site location and its associated seismic hazard, the selected design, and the risk of failure of the completed structure.

The objectives of the investigation are to establish controlling credible maximum and operating basis earthquakes and the corresponding ground motions for each and to assess the possibility of earthquake-induced foundation dislocation at the site.

5. SCOPE OF INVESTIGATION

To attain the objectives listed in Section 4, the investigations listed below shall normally be performed in the headworks area of run-of-river hydropower projects:

a. Geologic, seismologic, and neotectonic considerations.
b. Selection of earthquakes for analyses.
c. Factors influencing the selection of seismic evaluation parameters.
d. Selection of seismic evaluation parameters.

6. INVESTIGATION PLANNING

With the exception of fault evaluation studies, determination of seismicity and preliminary selection of the design earthquake are performed in conjunction with evaluation of the regional geology. Much of the data needed for describing the regional geology and for determining seismicity are identical, and therefore, the efforts can be combined. Engineering seismology requirements for in-depth studies of tectonic history, historical earthquake activity, and location of possible active faults are a logical extension of the regional geologic studies.

7. GEOLOGIC, SEISMOLOGIC AND NEOTECTONIC CONSIDERATIONS

The ground motions to be used in seismic analyses depend on the rate of activity of the regional seismic sources and on attenuation effects between the seismic sources and the site. Geologic and seismologic investigations provide the information needed to identify and characterize these seismic sources, including their earthquake potential, expected maximum earthquakes, frequency of occurrence of historic earthquakes of various sizes (earthquake recurrence intervals) and local attenuation characteristics.

7.1 Earthquake Potential

The approach to evaluating earthquake potential for the selection of seismic design or evaluation criteria shall be based on the premise that significant future ground shaking will be associated with active or capable faults, or with source zones of recognized seismic activity. Furthermore, the earthquake potential of distant faults as well as that of faults near the diversion structures shall be considered.

Evaluation of the earthquake potential of the region surrounding a headworks site requires the investigation of:

a. The regional tectonic framework, neotectonics, and local geologic setting.
b. Regional and local historical seismicity.
c. Potential seismic sources.
d. The relationship between event magnitude, fault characteristics, style of fault movement and ground motion estimates, for a given seismic source.

7.1.1 Regional Tectonic Framework, Neotectonics and Local Geologic Setting

7.1.1.1 Regional Tectonic and Neotectonic Framework

Geological, seismological and geophysical data related to regional tectonics and neotectonics shall be collected and interpreted in order to evaluate the regional tectonic framework. The evaluation may include:

a. Review of available literature, especially with regard to structural, tectonic, and neotectonic history;.
b. Interpretation of various types of imagery to identify regional structures of potential significance.
c. Reconnaissance mapping of the geology and geomorphology of the region.
d. Evaluation of the regional stress regime, utilizing in-situ stress measurements or focal plane solutions.
e. Analysis of geodetic data.

Different techniques shall be employed depending on the geologic setting and tectonic environment. In most cases, more than one approach may be used. Investigative techniques may include:

a. Compilation and review of existing information on the geology, seismology neotectonics, and regional tectonics.
b. Analysis of aerial photographs and other remote sensing imagery for lineation.
c. Geologic reconnaissance and mapping.
d. Geomorphic analysis.
e. Analysis of regional geophysical data (seismic reflection, seismic refraction, gravity and magnetics).
f. Field investigations of suspected fault traces (trenching, geophysical investigations, borings).
g. Age dating of geologic deposits (paleoseismic investigations).
h. Historical seismicity analysis, including review of local and regional earthquake sizes (intensity or magnitude), epicentral locations, focal depths, focal mechanism, aftershock patterns and magnitude-frequency distributions.
i. Monitoring of micro-seismicity in the area of the proposed project.
j. Correlation of seismicity and microseismicity with geologic structures.
k. Interpretation of regional stress regime from earthquake focal mechanisms, geologic indicators and geodetic data**.**

7.1.1.2 Local Geologic and Neotectonic Setting

Site-specific geologic and neotectonic information shall be gathered to ascertain some of the characteristics of the ground motion expected at the diversion structure site and to evaluate the potential for primary or sympathetic fault movement in the diversion structure foundation or at the location of other critical structures. Any geologic condition at or near the site that might indicate recent fault movement or seismic activity shall be thoroughly documented. Local geologic data shall be obtained through the review of literature, engineering reports of nearby projects, site inspections, and field explorations including material sampling and testing. Information may include:

a. Definition of type, extent, thickness, mode of deposition/formation and stability characteristics of rock units and soil deposits.
b. Location and chronology of local faulting, including amount and type of displacements estimated from historic and stratigraphic data, time of last rupture, rates of activity, strain rates, slip rates, etc. using appropriate measurement methods. In some cases the use of investigative techniques, such as trenching or age dating, may be required.
c. Interpretation of the structural geology including orientation and spacing of joint systems, bedding planes, and foliation; orientation and size of folds and intrusive and extrusive rock bodies, etc.

d. Determination of hydrogeological conditions, including the location of the water table, flow conditions, and underground water pressures.
e. Determination of foundation and abutment conditions.
f. Evaluation of potential abutment, reservoir, and other slope failures that may affect the safety of the facility.
g. Locate, if any, strong-motion records from historical earthquakes that occurred near the site or in areas with similar geologic or tectonic setting.

In general, when significant fault movement is accepted as a reasonable possibility in the diversion structure foundation or under other critical structures, the proposed headworks site may be abandoned and it shall be located at a less hazardous alternative.

7.1.2 *Historical Seismicity*

The review and evaluation of historical seismicity shall include consideration of the tectonic province in which the site is located, as defined by the regional tectonic evaluation, and of other potential earthquake sources that may be significant to the diversion structures. Published pre-instrumental and instrumental earthquake data shall be consulted. If needed, reports of earthquakes that may be significant to the selection of the controlling earthquakes shall be supplemented by detailed studies of the distribution of felt effects (intensity distribution). Emphasis shall be given to recorded data. Analysis of micro-earthquake records may assist in delineating the extent and geologic relationship of active faults, and assessing present-day tectonic stresses in a region. The comprehensive earthquake catalog developed by the National Building Code Development Project (BCDP, 1994) for Nepal and the surrounding area may be referred whenever applicable.

The completeness and accuracy of historical seismicity data varies from place to place. In some areas, pre-instrumental records may be partly or wholly absent. The wide spacing of instruments in the World-Wide Seismograph Network often results in incomplete or imprecise instrumental records. Further uncertainties, related to the insufficient number of installed instruments, may affect assigned epicenter locations and focal depth estimates. Such limitations can result in misleading, or even inappropriate, interpretations of the character and distribution of historical seismicity.

The instrumentally recorded earthquakes in the BCDP (1994) catalog are believed to be complete for the different magnitude earthquakes for the following periods:

M 6 and greater	Catalog complete for the period 1911 to 1992
M 5-1/2 and greater	Catalog complete for the period 1925 to 1992
M 5 and greater	Catalog complete for the period early 1960s to 1992
M 4-1/2 and greater	Catalog complete for the period late 1970s and early 1980's

7.1.3 Interpretation and Evaluation

7.1.3.1 Active or Capable Faults

Nepal is located at an active plate boundary. Seismic source evaluation shall focus on identifying active or capable faults and fault-related features, where large historic or prehistoric earthquakes have occurred, and zones (e.g., Sub-Himalayans, Lesser Himalayas, Higher/Tethian Himalayas).

For fault-like seismic sources, it may be possible to identify surface and subsurface features, and assess the following data:

a. Evidence of recurrent tectonic displacements; sense of slip (type of faulting).
b. Fault plane dip.
c. Width of past or potential ruptures.

d. Buried or blind thrusts, potentially associated with folds, that do not reach the ground surface.
e. Length and segmentation characteristics.
f. Slip rates.

7.1.3.2 Seismic Zones

The Seismic Hazard Mapping and Risk Assessment component of the National Building Code Development Project for Nepal (BCDP, 1994) shall be a good reference for seismic zones.

Analysis of the Earthquake Activity and the Tectonic Structure of Nepal

The study identified groups of earthquakes with major tectonic features leading to the identification and definition of seismic areas of assumed uniform seismicity. The three seismic zones (Figure 1) coincide roughly with the major tectonic provinces of Nepal and the surrounding region and they encompass the main northward dipping subduction zone under the region.

a. Seismic Zone B is the southern most of the three areas and roughly represents the shallow portion of the subduction zone beneath the Indo-Gangetic Plain.
b. Seismic Zone A encompasses both the Siwalik Hills and the Lesser Himalayas with their relatively high concentration of seismicity and it represents the moderate depth portion of the subduction zone characterized by great earthquakes.
c. Seismic Zone C is the northern most western zone in Nepal and represents the deeper portion of the subduction zone beneath the Higher/Tethian Himalayas. It also slightly lies in the southern most east part of Terai regions of Nepal.

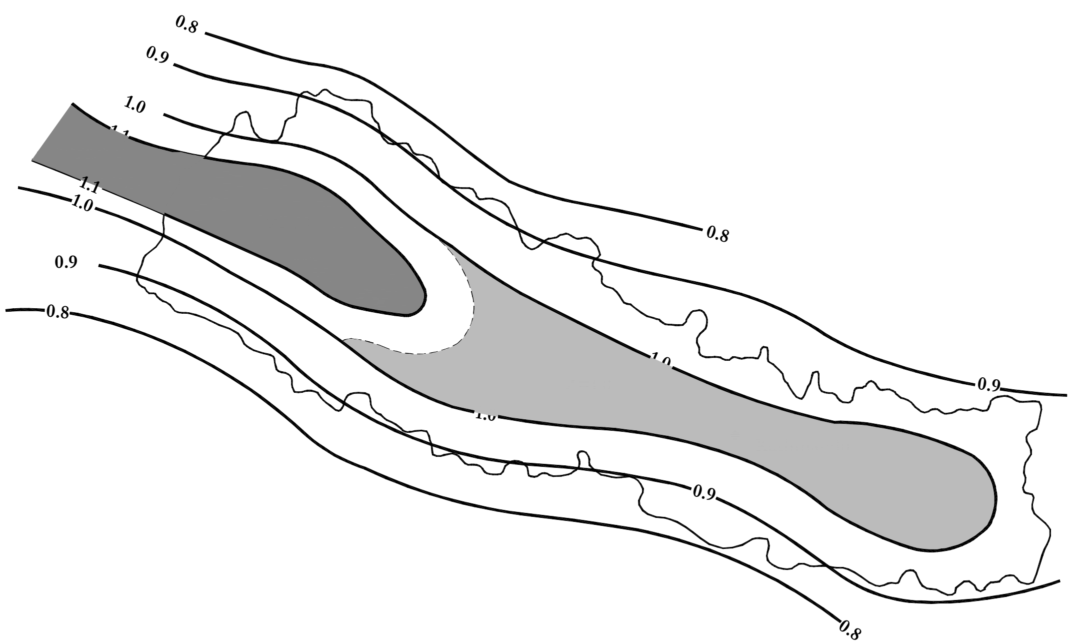

Figure 1: Seismic zoning map of Nepal (NBC 105: 1994)

The BCDP (1994) reported that the seismicity is spread over each of the zones in the approximate ratio 0.1:0.6:0.3 for Seismic Zones A, B, and C respectively. These source zones complement the seismicity of recognized active or capable faults. The maximum earthquake and the specified focal depth for the three seismic zones are as follows:

Seismic zone	Maximum magnitude (Ms)	Depth (km)
Northern, Zone C	8.0	60
Middle, Zone B	8.5	40
Southern, Zone A	7.5	10

(Source: A.M. Dixit et al, 1993)

One difficulty with the Seismic zone approach is the assessment of near-field effects, if the postulated earthquake is assumed to be centered in the immediate vicinity of the diversion structures. Near-field effects shall be recognized to be associated with active faults. Such effects shall include directivity (e.g., the intensity of motion is stronger in the direction perpendicular to the fault rupture plane than in the direction parallel to it) and can result in a "fling" in the recorded strong ground motions (presence of long period velocity pulses, related to the passage of a wave front). For seismic zones, separation of near-field effects from historical information used to quantify the earthquake of interest may be difficult and can result in distortion of the level of conservatism. Use of probabilistic methods may be used when this is the case.

7.1.4 Maximum Earthquake Magnitude

For both deterministic and probabilistic seismic hazard analyses, the maximum earthquake that each seismic source is capable of generating shall be estimated. These estimates shall be based on the physical characteristics of the source as well as the maximum historical earthquake.

Empirical relationships between earthquake magnitude and total fault length, rupture length, rupture area, maximum displacement per event, fault slip rate and seismic moment shall be referred. The use of these relationships to estimate maximum magnitudes requires the geologic assessment of a variety of parameters. Instrumental seismicity data and tectonic information derived from geophysical studies provide constraints on the physical dimensions, maximum magnitude and behavior of seismic sources.

Using multiple techniques leads to more reliable magnitude estimates than the use of any single technique.

In probabilistic seismic hazard analyses (PSHA), the uncertainty in maximum earthquake magnitude estimates shall be quantified. To arrive at a plausible distribution of maximum magnitude for each seismic source, the results of various techniques available to estimate this parameter can be aggregated.

After the earthquake potential of faults and of seismic zones in the region have been evaluated, including the potential maximum earthquake, it is possible to deterministically select a controlling earthquake(s). In some cases, earthquakes from both near sources and distant ones shall be considered, and to evaluate the effects of both short period and long period ground motion at the headworks site.

7.2 Earthquake Recurrence Assessment

For probabilistic seismic hazard assessments, the significant seismic sources shall be characterized by earthquake recurrence relationships that define the frequency of occurrence of earthquakes of various magnitudes, up to the maximum magnitude. Typically, either seismicity or geologic data, or both of them when available, are used to develop and constrain recurrence relationships.

Historical and instrumental seismicity data are useful for providing regional earthquake recurrence information. Because the length of the seismicity record is typically short relative to the return period of large earthquakes, it is usually insufficient to define source-specific recurrence characteristics, except for the most active faults. Techniques developed for checking earthquake catalog completeness and optimizing the use of older events in the catalog can be implemented to characterize seismically active sources. Insight may be obtained through a comparison with other sources in similar tectonic environments. Regional recurrence information derived from seismicity data can also be compared with that derived from geologic data.

Fault slip rates and strain rates are increasingly used to constrain earthquake recurrence for seismic hazard assessments.

7.3 Characterizing Ground Motions

After specifying the location and magnitude (or epicentral intensity) of each candidate earthquake and an appropriate regional attenuation relationship, the characteristics of vibratory ground motion expected at the site shall be determined. Vibratory ground motions have been described in a variety of ways, such as peak ground motion parameters, acceleration-time records (accelerograms), or response spectra. For the design of headworks structures, the controlling characterization of vibratory ground motion shall be a site-dependent design response spectra.

Elastic design response spectra of ground motions shall be defined by using standard or site-specific procedures. Elastic design response spectra represent maximum responses of a series of single-degree-of-freedom (SDOF) systems to a given ground motion excitation. The response spectrum amplifications depend on the values of damping and are significantly influenced by the earthquake magnitude, source-to-site distance, and the site conditions

7.3.1 Standard or Normalized Response Spectra

The development of standard response spectra for various soil profiles at specified return periods shall start with the spectral acceleration ordinates obtained from hazard maps.

7.3.2 Site-specific Design Response Spectra

Site-specific response spectra correspond to those expected on the basis of the seismological and geological calculations for the site.

While the deterministic method provides a single estimate of the peak ground acceleration and response spectral amplitudes, the probabilistic method estimates these parameters as a function of probability of exceedance or return period.

Wherever possible, site-specific design response spectra shall be developed statistically from response spectra of strong motion records of earthquakes that have similar source and propagation path properties as the controlling earthquake(s) and are recorded on a foundation similar to that of the diversion structures. Important source properties include magnitude and, if possible, fault type and tectonic environment. Propagation path properties shall include distance, depth, and attenuation. As many accelerograms as possible that are recorded under comparable conditions and have a predominant frequency similar to that selected for the design earthquake shall be included in the development of the design response spectra. Also, accelerograms shall be selected that have been corrected for the true baseline of zero acceleration, for errors in digitization, and for other irregularities.

Where a large enough ensemble of site-specific strong motion records is not available, design response spectra shall be approximated by scaling that ensemble of records that represents the best estimate of source, propagation path, and site properties. Scaling factors can be obtained in several ways.

a. The scaling factor may be determined by dividing the peak or effective peak acceleration specified for the controlling earthquake by the peak acceleration of the record being rescaled. The peak velocity of the record shall fall within the range of peak velocities specified for the controlling earthquake, or the record shall not be used.

b. Spectrum intensity can be used for scaling by using the ratio of the spectrum intensity determined for the site and the spectrum intensity of the record being rescaled.

c. Acceleration attenuation relationships can be used for scaling by dividing the acceleration that corresponds to the source distance and magnitude of the controlling earthquake by the acceleration that corresponds to the source distance and magnitude of the record being rescaled.

Because the scaling of accelerograms is an approximate operation at best, the closer the characteristics of the actual earthquake are to those of the controlling earthquake, the more reliable the results. For this reason, the scaling factor shall be held to within a range of 0.33 to 3 for gravity dams.

Site-dependent response spectra developed from strong motion records shall have amplitudes equal to or greater than the mean response spectrum for the appropriate foundation anchored by the PGA determined for the site. This minimum response spectrum may be anchored by an effective PGA determined for the site, but supporting documentation for determining the effective PGA will be required.

A mean smooth response spectrum of the response spectra of records chosen shall be presented for each damping value of interest. The statistical level of response spectra used shall be justified based on the degree of conservatism in the preceding steps of the seismic design process and the thoroughness of the development of the design response spectra. If a rare event is used as the controlling earthquake and the earthquake records are scaled by upper bound values of ground motions, a response spectrum corresponding to the mean of the amplification factors shall be used if the response spectrum is based on five or more earthquake records.

7.3.3 Accelerograms for Acceleration-time History Analysis

Accelerograms used for dynamic input shall be compatible with the design response spectrum and account for the peak ground motions parameters, spectrum intensity, and duration of shaking. Compatibility is defined as the envelope of all response spectra derived from the selected accelerograms that lie below the smooth design response spectrum throughout the frequency range of structural significance.

8. FACTORS INFLUENCING SELECTION OF SEISMIC DESIGN PARAMETERS

The design objectives and possible modes of failure of headworks structures control the analysis requirements and, therefore, the way in which seismic design parameters shall be selected and specified. Various methods of analysis call for different ways of specifying earthquake motions, for a given earthquake level. Therefore, effective communications shall be established between the geologists, seismologists and engineers responsible for specifying the earthquake motions, and the engineers who will analyze the headworks structures for those motions. Factors that affect the specification of seismic parameters include:

a. Seismic hazard rating of the headworks site.

b. Risk rating of the completed structures.

c. Type of dam and its possible mode(s) of failure.

Judgment and professional experience shall be employed to determine the appropriate methodology for analyzing the headworks structures and specifying the seismic parameters, based upon the above factors.

8.1 Influence of Site Hazard Rating and Structure Risk Rating

8.1.1 Site Hazard Rating

Because of their geographical location, some sites are more exposed than others to potential earthquake ground motions. The site hazard rating, defined below, shall be used to quickly differentiate between several sites. The seismic hazard rating of a headworks site influences the evaluation requirements and the level of refinement necessary for the definition of seismic design parameters. The seismic hazard of a site can be rated based on the PGA that would be expected from the MCE and considerations such as the nearby presence of active faults. A preliminary rating of the site shall also be made using existing seismic zone maps.

The following table, which applies to the dam location regardless of the type of dam, can be used to rate the seismic hazard of a headworks site. This table provides a quick way of rating the seismic hazard. Other factors shall be considered before making the final decision regarding the level of effort required for the seismic design of headworks structures. Sites in Nepal will be in Hazard Class III or IV based on the maximum earthquakes given for the zones by the BCDP (1994; Zone 1, Ms 7.2, Zone 2, Ms = 8.0, Zone 3, Ms = 7.6).

Table 1: Seismic hazard rating

Conditions	Hazard class (hazard rating)
PGA < 0.10 g	I (Low)
0. 10 g ≦ PGA < 0.25 g	II (Moderate)
PGA ≧ 0.25 g, but no active fault within 10 km of site	III (High)
PGA ≧ 0.25 g, active fault closer than 10 km from site	IV (Extreme)

The above table applies to sites where good foundation materials are present. It would be prudent to upgrade the hazard class by one unit if questionable materials, such as low density saturated silts and sands or other potentially loose deposits, are present or suspected in the foundations of the headworks structures.

The Hazard Class of a headworks site will provide a preliminary indication of the seismic evaluation requirements:

a. For sites in Hazard Class I, it will normally be sufficient to define seismic design parameters with peak ground motion values. The simplest methods of analysis shall be acceptable. Most dams in this hazard class will not experience damage under the condition, and consideration of OBE will not be necessary if the dam is checked for the MDE.

b. For sites in Hazard Class II, seismic design parameters may be defined with peak ground motion values, response spectra, or acceleration time histories, depending on the type of dam, its risk rating and its possible mode(s) of failure, discussed subsequently. Consideration of the OBE shall not be necessary as well-designed dams in Hazard Class II shall be capable of resisting the MDE with little damage, if any at all.

c. For sites in Hazard Class III, seismic design parameters shall preferably be specified by acceleration time histories, although response spectra may be sufficient for the evaluation of some concrete dams and appurtenances structures. Separate consideration of OBE will often be required.

d. For sites in Hazard Class IV, acceleration time histories are mandatory to specify and represent fault-specific phenomena such as near-field or directivity effects, and to account for potentially critical foundation conditions.

8.1.2 Structure Risk Rating

The potential risk associated with dams consists of structural components and socioeconomic components. The structural components of risk depend primarily on storage capacity and on the height of the dam, as the potential downstream consequences are proportional to the mentioned values. Socio-economic risks can be expressed by the number of persons who need to be evacuated in case of danger and by potential downstream damage.

The two following tables shall be used to rate the risk associated with dams in Nepal. Four risk factors are separately weighted as low, moderate, high or extreme:

Risk Factor	Extreme	High	Moderate	Low
Contributions to risk (weighting points)				
Capacity (hm^3)	> 120 (6)	120-1 (4)	1-0.1 (2)	<0.1 (0)
Height (m)	> 45 (6)	45 – 30 (4)	30 – 15 (2)	<15 (0)
Evacuation Requirements (No. of persons)	1000 (12)	1000 – 100 (8)	100-1 (4)	None (0)
Potential Down-stream damage	High (12)	Moderate (8)	Low (4)	None (0)

Hazard potential classification for dams shall be based on:

Hazard potential classification	Loss of Human Life	Economic, Environmental, or lifeline Losses
Low	None expected	Low and generally limited to dam owner
Moderate	None expected	Yes
High	Probable, one or more expected	Yes, but not necessary for this classification

(Source: USACE, ER 1110-2-1806, 1995b)

The weighting points of each of the four risk factors, shown in parentheses in the table, are summed up to provide the Total Risk Factor as:

Total Risk Factor = Risk Factor (Capacity) + Risk Factor (Height) + Risk Factor (Evacuation Requirements) + Risk Factor (Potential Downstream damage)

The Risk Class of the dam is assigned based on the Total Risk Factor, as follows:

Total Risk Factor	Risk Class (Risk Rating)
(0 – 6)	I (Low)
(7 - 18)	II (Moderate)
(19 - 30)	III (High)
(31 - 36)	IV (Extreme)

The risk classification of the dam is needed to further guide the selection of seismic design parameters, as dams with high-risk rating will normally require a sophisticated level of evaluation.

Dams in Risk Classes III or IV will require a detailed method of analysis and the use of acceleration time histories, especially if such dams are also associated with a high site hazard rating. Simpler methods of evaluation, using response spectra or peak ground motion parameters, may be acceptable for dams of low or moderate risk ratings.

For dams in Risk Class I, it is acceptable that the MDE represents a level of motion less than that which would be induced at the dam site by the CMCE. The MDE shall be derived from the CMCE for any dam at least 15 m high or impounding a reservoir of 12.34 hectare-meter (100 acre-feet capacity), or more, where loss of life or substantial downstream damage could occur as a consequence of failure.

8.2 Influence of Type of Dam

The type of dam and the possible mode(s) of failure must be considered along with the hazard rating to finalize the selection of seismic evaluation parameters. Different approaches are required for either concrete or embankment dams**.**

8.2.1 Concrete Dams

Foundation-structure interaction effects can be quite significant for concrete dams sited upon relatively flexible bedrock and/or built across relatively narrow valleys. When this is the case, the foundation needs to be included in the mathematical model used for the analysis. Selection of the foundation properties is critical to the analysis. Such requirement usually calls for field investigations, including measurement of the shear and compressive wave velocities of the subsurface foundation materials.

Both dynamic foundation-structure and fluid-structure interaction effects tend to lower the natural frequencies of vibration of concrete dams. This shall be considered when assessing the appropriateness of the frequency content of acceleration time histories that may be used in the analyses.

Peak or peak effective ground motion parameters, and response spectra are usually sufficient to define seismic criteria for simplified methods of analysis of concrete dams. Response spectra shall normally be smoothed to eliminate peaks or valleys that are not representative of meaningful statistical trends.

Dynamic finite element analysis shall use either response spectra or acceleration time histories. If the assumption of linear-elastic behavior is acceptable, as is commonly the case, either appropriate response spectra or time histories shall be used to compute earthquake-induced stresses in the headworks concrete. However, if a nonlinear response is anticipated, or if the number of stress cycles or the extent of overstressed zones is likely to be significant, time history methods need to be used.

If the headworks structure can be adequately represented by a 2-D model, two components of motion are sufficient, one vertical and one horizontal. For concrete arch dams and most curved gravity dams, two orthogonal horizontal and the vertical component of motion, preferably from the same recording station and earthquake event, shall be used in the dynamic analysis.

8.2.2 Embankment Dams

Analysis requirements for embankment dams shall be governed by the expected modes of response and possible failure modes.

For the dynamic analysis of an embankment dam, the ground motion may be characterized in terms of a magnitude and distance-dependent response spectrum. In simplified procedures, this spectrum may be used in combination with other parameters, estimated separately, such as the number of equivalent stress cycles. When the foundation and/or embankment materials are susceptible to significant loss of stiffness or strength, a dynamic time history analysis shall be used to obtain the number and amplitudes of induced stress-strain cycles. These data are then used to establish whether the shaking could induce large soil deformation or liquefaction. The response spectrum will serve, however, to develop appropriate time histories for the time history analysis.

For most embankments, the vertical component of motion does not need to be included in the analysis. In most cases, one horizontal component is sufficient to compute the response of an embankment dam, if represented by a plane-strain 2-D model.

9. SELECTION OF SEISMIC EVALUATION PARAMETERS

The seismic evaluation parameters to be used for seismic design of diversion structures shall represent one or several ground motion related variables or characteristics, such as peak ground acceleration, velocity or displacement values, response spectra or acceleration time histories that will characterize the MDE or OBE. These variables may be selected deterministically, or based on a PSHA.

Various combinations of these parameters are often used. Several acceleration time histories may define the same MDE. An OBE can be represented by peak ground acceleration (PGA) or by a specified spectral shape, depending on the type of evaluation required.

The seismic evaluation parameters representing the MDE or the OBE serve as input data for the numerical analyses of diversion structures and its principal appurtenant facilities, such as intake. The results of such numerical analyses shall be used to evaluate performance and safety of diversion structures, given the postulated level of ground motion.

Source, transmission path, and local conditions influence ground motion at any given site.

a. Source effects include fault type; rupture dimensions, mechanism and direction; focal depth; slip distribution and rate; stress drop; and amount of energy released;
b. Transmission path effects relate to geometric spreading, absorption, refraction or reflection of earthquake waves as they travel away from the source. Transmission path effects may also include phenomena due to the influence of bedrock type and crustal inhomogeneities, the presence of deep alluvium or basin boundaries, and to wave propagation effects (e.g., direction of wave travel versus direction of fault rupture).
c. Local effects result from the topographic and geologic conditions present at the diversion structure site, and from the possible interaction between structures and the surrounding foundation materials. Local effects may be enhanced by the relative orientation of the dam axis compared to the orientation of the causative fault rupture surface.

The factors generally considered the most significant to the specification of seismic evaluation parameters are:

a. Site classification (alluvium or rock).
b. Physical properties and thickness of foundation materials.
c. Closeness to the causative fault (near-field and directivity effects).
d. Distance from the zone of rupture (zone of energy release).
e. Source mechanism (strike-slip, thrust, normal, etc.).
f. Magnitude of the design event.
g. Tectonic framework (subduction).

Other factors, such as direction of fault rupture propagation, type of faulting (normal, reverse or strike-slip) and topography, are significant and increasingly being included in seismic studies. Thrust fault ruptures tend to generate substantially stronger shaking (a comparative increase of perhaps as much as 30 percent) than strike-slip or normal earthquake events. Normal-slip earthquakes (extensional fault regime) may generate less severe earthquake motion than the two other principal types of fault movement. Reflections of seismic waves at basin boundaries potentially contribute to increasing the intensity of shaking at some sites**.** Where thrust faulting represents a specific concern the development of seismic parameters shall be based on fault specific data. Judgment and conservatism shall be applied, if the seismic criteria are to be derived from general considerations.

Preferably, seismic design parameters shall be specified by using site-dependent considerations, making use of existing knowledge and actual observations that pertain to earthquake records obtained from sites with similar characteristics. When applicable site data are too scarce to be meaningful, non-site-specific ground motion may be used, or additional data shall be collected. The most frequently used seismic design parameters are described in the following sections.

9.1 Peak Ground Motion Parameters

Ground motion can be simply quantified by peak values of expectable acceleration, velocity and/or displacement. Empirical relationships, called attenuation equations, shall be derived from the interpretation of available strong motion records and relate peak ground motion parameters to magnitude and distance from the source of energy release. Attenuation equations are sensitive to the estimates of distance and magnitude, especially in the near-field. The scatter between observed and predicted values is often significant, as many factors (including but not limited to source effects, site characteristics and the conditions of placement of the recording instruments) affect strong motion records.

Peak ground acceleration (PGA) often represents the main seismic evaluation parameter for simplified diversion structure analysis purposes. The peak ground acceleration (usually as a fraction of the peak) is the earthquake ground motion parameter usually used in the seismic coefficient method of analysis. Many PGA attenuation equations have been developed in recent years. These guidelines are not intended to recommend any specific attenuation relationship(s) to estimate the PGA or other peak ground motion parameters for Nepal.

Since Nepal has little data on which to base derivation of specific attenuation functions, the following table provides a partial listing of published attenuation functions that may be useable along with the geographic and tectonic environments where they are applicable. Such references are frequently updated. The user is advised to check for the most recently applicable sources of information on the subject.

Applicability	Fault Type	Reference
Subduction Zones	Oblique Thrust	Crouse et al., 1988
Subduction zones	Oblique Thrust	Youngs et al., 1997
Subduction Zone – Japanese Events	Thrust	Kawashima et al., 1984'
Cascadia Region	Oblique Thrust	Atkinson and Boore, 1997
Western USA	Undifferentiated	Joyner and Boore, 1982
Western USA	Undifferentiated	ldriss, 1991
Eastern USA	Undifferentiated	EPRI, 1988
Central/Eastem USA	Undifferentiated	Atkinson and Boore, 1995
Central/Eastem USA	Undifferentiated	Toro et al., 1997

These relationships were found applicable for Nepal in the studies by the National Building Code Development Project (BCDP, 1994). Attenuation model of Youngs et al (1997) used in development of seismic hazard map of Nepal by M.R. Pandey et al (2002) is

$$\mathrm{In}(PGA) = 0.2418 + 1.414M - 2.552\mathrm{In}(r_{rup} + 1.7818e^{0.554M}) + 0.00607H$$

Where

M = moment magnitude

r_{rup} = closest distance to rupture (km)

H = depth (km)

Care shall be taken to study the background material for each reference to assure that it is applicable to the regional and site geology under consideration. When several relationships are considered applicable, it is often appropriate to apply some type of averaging, such as a weighted average, to the calculated values. Care shall be exercised before using any particularly high or low estimate. In any event, considerable judgment is required to select an attenuation relationship for ultimate use.

It is generally desirable to first define the PGA as that occurring on "outcropping bedrock" (most of the above references provide attenuation equations applicable to either bedrock or one or several soil types, in free-field condition). It shall then be adjusted, as required, to account for unusual site conditions, such as deep alluvium or soft sites, where expectable accelerations generally contain more energy in the long period range than those on a rock site. The seismic hazard map of Nepal (Figure 2) produced by Department of Mines and Geology provided below may be referred wherein peak ground acceleration at bedrock that has a 10% probability of being exceed over 50 years (return period of 500 years).

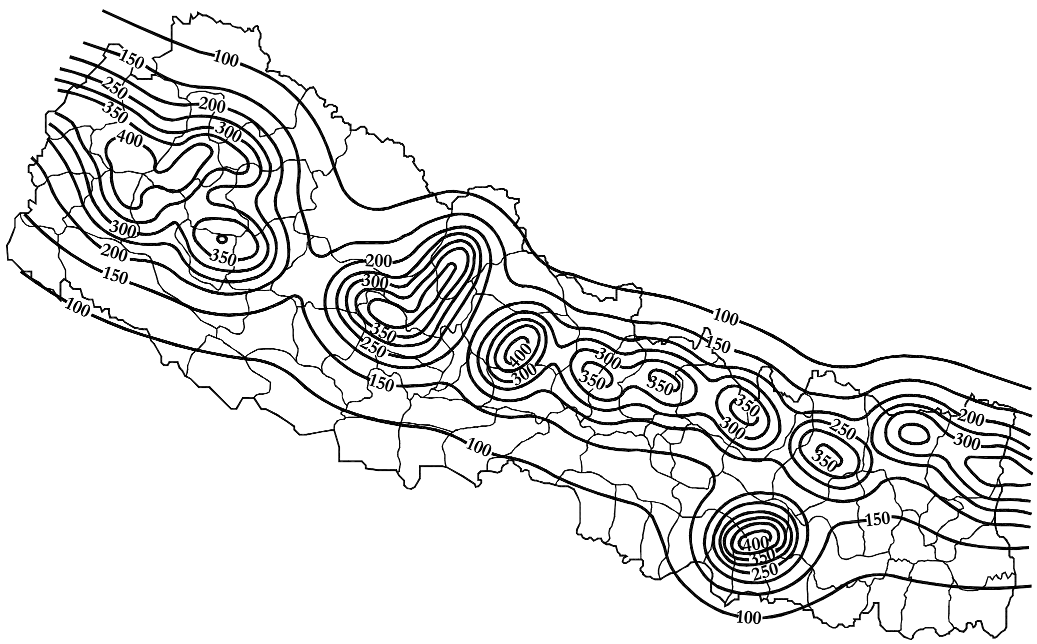

Figure 2: Seismic hazard map of Nepal (Pandey *et al* 2002)

For peak acceleration on the rock approximately up to 0.15g, the peak accelerations on the top of cohesionless soil deposits and stiff soils are slightly more. For peak acceleration on the rock > 0.15g the peak acceleration on the top of cohesionless soil deposits and stiff soils slightly small. In soft soil sites, for peak acceleration on the rock up to 0.40g, the accelerations on soil surface are going to be more. Finally, in soft sites, for peak acceleration on the rock above 0.40g, the acceleration on soil surface are going to be not as large as of bed rock.

It shall be noted that, while outcropping bedrock ground motion may be used directly as seismic input to analyze a small dam, PGA and other ground motion parameters considered for a large dam, especially if founded on alluvium, may differ from the outcropping bedrock condition. The motion below the main section of an embankment dam may substantially differ from that recorded at its abutments.

The error term associated with attenuation equations and the statistical significance of the predicted values (absolute maximum, effective, mean, median, or mean plus one standard deviation) shall be carefully reviewed and understood prior to using them for analytical purposes. It is preferred used mean, rather than extreme values of the PGA.

With more reliable attenuation equations being now available for spectral amplitudes, it has become preferable to select a site-dependent PGA, as well as spectral ordinates, based upon these attenuation relationships. In some applications, mean plus one standard deviation PGAs are deemed more appropriate, based on risk rating and degree of source activity.

It shall be noted that peak ground velocity (PGV) probably better quantifies ground motion through a single parameter than PGA, especially in the near-field. So far, few attenuation relationships have been developed for PGV, or for peak ground displacement (PGD).

Detailed evaluation of the vertical PGA is recommended for near-field sites. Recorded evidence indicates that close to the fault rupture and, especially, in the case of thrust faulting, peak vertical accelerations may equal or even substantially exceed peak horizontal accelerations. While such observations are of limited significance in the case of most earth dams, reliable specification of the vertical PGA is required for concrete dams and for headworks structures. Unfortunately, few attenuation relationships are currently available for the vertical component of ground motion. For intermediate- or far-field locations, the vertical PGA may be conservatively taken as two-thirds or one-half of the horizontal PGA, respectively. The vertical component of short period sites approaches higher fraction of the horizontal ground motions.

9.2 Duration

The duration of shaking is a most significant seismic design parameter. It has been directly related to the extent of damage, especially in the case of embankment dams. The buildup of excess pore pressures in the saturated part of an embankment or -in a foundation alluvium is directly related to both the intensity and the duration of shaking.

The duration of earthquakes shall be estimated in different ways. Of significance to engineers are the bracketed duration (duration of strong shaking), measured between the first and the last occurrence of acceleration pulses greater than 0.05 g, at frequencies above 2 Hz and the Husid's duration. Husid's duration relates to the total amount of energy contained in a specific record. The effective duration is defined as the length of shaking during which 90 percent of the energy content of the motion has been generated.

Local conditions may affect the expected duration of earthquake shaking and shall be considered on a case-by-case basis. Sedimentary basins filled with soft soils are susceptible to considerable increase in the effective duration of shaking, as a result of wave amplification and multiple reflections along the basin boundaries. This is called “basin effects”. In complex tectonic environments, consideration shall also be given to possible increase in the duration of shaking as a result of multiple or sympathetic fault raptures.

9.3 Response Spectra

Response spectra can be used directly as seismic input in some cases (e.g., concrete dams), but are also increasingly used to develop or to rate the adequacy of acceleration records used for design of headworks structures.

9.3.1 Generalized Spectral Shapes

Generalized response spectra are most useful at the conceptual or preliminary phase of a new project, or to perform initial evaluations of existing diversion structures. When generalized spectral shapes are used, they shall be checked, whenever possible, for consistency with historical earthquake response spectra obtained for site conditions and earthquake magnitudes similar to those expectable at the site considered.

Spectral shapes are generally provided in a normalized format (i.e., scaled to g), and for five percent damping. In order to define seismic input motion, they are scaled uniformly (independently from the period of vibration) to a specified PGA (in most cases), or another applicable earthquake parameter (e.g., spectral intensity). Care shall be exercised to minimize the extent of scaling. In the case of the MDE, mean-plus-one-standard-deviation normalized spectral shapes have often been scaled to a specified mean PGA, in order to avoid excessive conservatism in the specification of the seismic input. If damping higher than five percent is expected, further scaling shall be done.

9.3.2 Response Spectra Based upon Attenuation Relationships

9.3.2.1 Horizontal Response Spectra

Current emphasis in the diversion structure engineering profession has been to give added credibility to site-dependent response spectra. Indeed, for major new dam projects, the recommended alternative to generalized spectral shapes consists of using reliable attenuation relationships for spectral amplitudes. In that manner, one can develop spectral envelopes that, as a minimum, are both magnitude- and distance-dependent.

Early attenuation equations for spectral amplitudes (McGuire, 1977; Hanks and McGuire, 1981; Katayama, 1982; and Campbell, 1983) defined spectral response parameters only as a function of magnitude and distance. In addition to being magnitude- and distance-dependent, more recent (and more reliable) spectral attenuation relationships account for various general site conditions (soft soil, hard soil or rock), tectonic environments (crustal or subduction events), and fault rupture types (strike-slip, thrust, or normal).

Most of the current research on the attenuation of ground motion includes sets of equations for both PGA and spectral amplitudes at selected periods. These coefficients are applicable to the definition of ground motion spectral characteristics in free-field conditions.

As in the case of PGA, it is desirable not to rely upon a single set of spectral attenuation equations to develop a suitable response spectrum. Each relationship, as defined in the above references, may have restrictions in its range of application. Some are biased in the near-field or in the far-field, due to the fitting procedure used in their development. Therefore, given the applicable site conditions, distance and magnitude of the MDE or OBE , several sets of equations shall preferably be used.

A conscious effort shall be made to select spectral parameters based upon relationships that best fit the characteristics of the site and area of interest. One shall carefully review the shapes and amplitudes of the generated spectra. If there is no restriction to use one specific relationship, one may use the most conservative of several relationships or, better, apply some averaging scheme (e.g., a weighted average) to the results obtained from several sets of equations. It shall be noted, however, that the averaging process for spectral amplitudes at specified periods is slightly more complicated than in the case of the PGA. Because the various attenuation relationships for spectral coefficients available from the literature have not been developed for the same set of periods, a suitable interpolation procedure (e.g., geometric averaging) must be implemented to compare or average the calculated spectra at the desired reference periods.

For dams built on alluvial foundations, it is generally acceptable to use attenuation relationships that have been developed for stiff soil sites. Shear wave velocity of the substratum may be used as a means to differentiate between site conditions. Hence, different attenuation equations will apply to different average shear wave velocities.

9.3.2.2 Vertical Response Spectra

Vertical response spectra may be simply scaled from the horizontal spectra. Such scaling shall be done using a reduction factor in the range of 1/2 to 2/3, applied to all frequencies (or periods).

Some attenuation relationships have now been developed for vertical response spectra, using statistical regression analyses of recorded vertical earthquake motions. These analyses provide a better means for developing seismic parameters applicable to vertical earthquake motion. It shall be noted that, in the near-field, vertical spectral amplitudes may be about the same, or may even exceed the corresponding horizontal spectral amplitudes, based on recent earthquake records.

9.3.3 Damping and Response Spectra

The selected damping and the number of damping values for which response spectra shall be developed to represent the MDE or OBE shall encompass a range of values that applies to the type of headworks structures and ground motion level considered. For example, damping values typically considered for the analysis of concrete dams range from 3 to 10 percent, depending on the severity of the motion to be represented and on whether the response is expected to be predominantly elastic or non-linear. Non-linearities, such as cracking of the concrete, can be indirectly included in the analysis by assuming a higher level of damping. Damping values for the analysis of embankment dams range from 5 to 20 percent, and are a function of the seismically induced shear strain levels in the embankment materials.

For the purpose of characterizing ground motion and comparing the damage potential of various earthquake records, five percent of critical damping is the value most commonly used to compute response spectra. Generalized spectral shapes and spectral amplitude attenuation relationships are also typically developed for that value.

If response spectra are needed for damping values other than five percent, for example, to evaluate the post-cracking response of concrete structures, or if severe shaking is considered, scaling procedures, such as developed by Newmark and Hall, (1982), are recommended. In these procedures, the spectral amplitudes at damping values other than five percent are obtained by multiplying the spectral amplitudes at five percent by scaling factors. The scaling factors depend on both the new damping and the frequency (or period) considered. Similar scaling factors can also be used to approximately represent a level of risk different from that of the specified response spectrum, without going through the cumbersome process of developing additional response spectra.

9.4 Acceleration Time Histories

The definition of seismic parameters by peak ground acceleration or peak ground velocity and spectral shapes is sufficient for many headworks structures. However, in some cases, such as for high hazard dams, time history analysis shall be required. Stresses approaching or exceeding the strength of the headworks structures or foundation materials, or the need to obtain information on the severity and extent of inelastic demands, may dictate the use of at least linear-elastic time history analysis. If nonlinear analysis is contemplated, time histories are also required. Time history analysis is needed to calculate deformations in embankment dams with a greater level of confidence.

When it is appropriate to perform time-dependent response analysis, it is recommended that several sets of acceleration time histories be used to represent the MDE or OBE, especially if natural records are being used. Each set is composed of one, two or three components of motion, in mutually orthogonal directions, depending on the analysis requirements. Typically, four or five separate sets are required. This is because natural time histories may include insufficient energy content at some of the frequencies of interest to the diversion structure being investigated, and would result in unconservative analysis.

Nonlinear response analyses of embankment dams, which are becoming increasingly more common, are stress-path dependent. This dictates more thorough analyses than for linear-elastic or equivalent-linear procedures. Different acceleration time-histories, scaled to consistent amplitudes and duration of shaking in the purpose of representing a same MDE or OBE , may lead to different results in a nonlinear analysis framework. Hence, the use of a single set of input accelerograms may be insufficient to properly characterize the dynamic response of headworks structures to postulated earthquake motion.

In some cases, if an acceleration time history is successively used as seismic input with normal or reverse polarities (all acceleration data points have been multiplied by minus one), the pattern of final non-recoverable deformations may be affected. Some headworks structures may experience their largest non-recoverable deformations in different directions (toward upstream or downstream), depending on the specific characteristics of the input accelerograms. It is also recommended that both normal and reverse polarities be tried successively, for each set of acceleration time-histories, to ensure that no potentially critical seismic loading condition has been overlooked.

The following paragraphs summarize current acceptable procedures to develop or modify time histories for the purpose of dam analysis.

9.4.1 Overview of Spectrum-Compatible Time History Development Procedures

Given a prescribed target response spectrum, an associated controlling earthquake, and a set of ground motion parameters, one can use the methods described in the following pages to develop ground motion time histories compatible with such target response spectrum. Only Method I provide close matching with the target spectrum. The other methods, however, can provide histories with response spectra that reasonably match the target spectrum in the range of periods of interest.

9.4.1.1 Response-Spectrum Matching Time History Adjustments: Method 1

This method, as generally practiced, starts with the selection of a suitable three-component set of initial or "starting" accelerograms. Then, the method proceeds to adjust each component iteratively, using either a time-domain or a frequency-domain procedure. The iterative process continues until a close match is achieved with a specified target spectrum and other associated parameter values.

The time-domain adjustment procedure usually produces only small local adjustments to the selected time histories, thereby producing response-spectrum-compatible (RSC) histories closely resembling the initial motions. Thus, the general phase characteristics of the seismic waves in the starting histories are largely maintained, while achieving close compatibility with the target spectrum.

The commonly used frequency-domain procedure retains the phase relationships of an initial motion, but does not always provide as close a fit to the target spectrum, as does the time-domain procedure, or requires much iteration. Also, the motion produced by the frequency-domain procedure often shows greater visual differences from the initial motion.

9.4.1.2 Source-to-Site Numerical Model Time-History Simulation: Method 2

Descriptions of this method are found in Papageorgiou and Aki (1983); Silva and Lee (1987); Bolt (1987); Boore and Atkinson (1987); and Somerville and Helmberger (1990). This simulation generally starts by constructing a numerical model to represent the controlling earthquake source and source-to-site transmission and scattering functions. Then, accelerograms are synthesized for the site, using numerical simulations based on various plausible fault-rupture scenarios or on the assumption of point sources. Each individual accelerogram obtained through this simulation procedure generally does not produce a time history having its spectrum and other characteristics closely matching the target parameters. Even when applying certain constraints on the modeling parameters, this method often requires a large number of simulations to achieve reasonable consistency with the target values, on an ensemble average basis.

The method is useful for assessing various earthquake scenarios and for examining the effects of various seismologic parameters on the resulting ground motions. Of particular interest, the procedure can produce a time history with a specific desired characteristic, such as a large velocity pulse or "fling", e.g., as would be expected for a near-source site. Therefore, it can be used to generate suitable starting motions for use in Method 1. Because numerous time history simulations are generally required to achieve a "stable" average spectrum for the ensemble, this method is often not practical for developing multiple sets of time histories for direct engineering application. Rather, it is used for developing a set of time histories to be used directly, or to supplement a set of recorded accelerograms in developing site-specific target response spectra and associated ground motion parameter values.

9.4.1.3 Multiple Natural Time-History Scaling

This method is described in the Standard Review Plan of the U.S. Nuclear Regulatory Commission (1989), and in the 1984 UBC. It starts by selecting multiple (generally seven or more), three-component sets of actually recorded accelerograms. These are subsequently scaled in such a way that the average of their spectral response ordinates matches the target spectrum over the specified frequency (or period) range of interest. Experience in applying this method show that its success largely depends on the initial selection of time histories. Because of the frequent scarcity of suitable time-histories, individual accelerograms often shall be scaled up or down by large multiplication factors (sometimes up to 2.5), although smaller scaling factors are desirable.

When scaling from real records, it is preferable to scale downward, rather than upward. The possible need to either use large scaling factors or to scale upward raises questions about the appropriateness of such method. Experience also indicates that, unless a large ensemble of time histories (typically 20 or more) is selected, it will be difficult to achieve compliance with the target spectrum over the entire range of interest.

9.4.2 Selection of Starting Time Histories

The following guidelines shall be used when selecting a "starting" time history:

a. Preferably, each set of starting accelerograms shall be selected from sets of natural accelerograms, recorded for several past seismic events with similar fault mechanism. Each set shall possess a ground motion pattern significantly different from those of the other selected sets.

   b. Where possible, each past seismic event selected shall have a magnitude within plus or minus 0.5 of the target magnitude of the controlling event.

   c. Generally, the distance between the strong motion stations that recorded acceleration histories considered as "starting" motions and their causative earthquake source shall be within 10 km of the target source-to-site distance.

   d. The selected natural accelerograms shall have their PGA, PGV, PGD and their durations of strong shaking within a range of -25 percent to +50 percent of the corresponding target values.

e. The selected accelerograms shall be free-field recordings on rock, rock-like or stiff soil conditions that reasonably match the control point conditions. No recordings on soft soils shall be used.

f. For a near-field controlling seismic event, e.g., within about 10 km or less of the site, the selected accelerograms shall contain a large velocity pulse or "fling". If suitable records are not available, Method 2 can be used to generate a starting set of time histories with a "fling". Alternately, recorded motions shall be modified to include a large directional velocity pulse obtained at another site.

9.4.3 Guidelines for Generation of Response-Spectrum-Compatible Time Histories

The set of recorded time histories selected to represent a starting motion shall be made response-spectrum compatible (RSC), using the previously described Method 1, or shall have a suitable frequency content in the frequency range of interest to the headworks structure being analyzed. Five percent damping response spectra are generally used for this purpose. Generation of the RSC histories shall conform to the following guidelines:

a. For each principal component of motion, response spectrum matching shall be carried out for at least 75 spectral frequencies (or periods), over the frequency range 0.1 to 20 cps (or period range, 0.05 to 10 s). These frequencies (periods) shall be near-equally spaced, when plotted with a logarithmic scale.

b. Once each RSC acceleration time history is generated, its spectral values shall be computed for the minimum number (75) of specified spectrum-matching frequencies (or periods). These values shall be compared with the corresponding target response spectral values.

c. Each generated RSC history shall preferably have its spectral amplitudes within minus five percent to plus ten percent of the target values, for the corresponding frequencies (periods).

d. Whenever possible, the peak values of each generated acceleration history peaks near the target values. They shall be checked to ensure that they do not have numerically-generated baseline drifts. Any baseline drift shall be corrected using an appropriate baseline correction procedure, such as by introducing an acceleration impulse, by high-pass filtering, or by using another accepted method.

e. Each generated acceleration history shall also be checked for frequency contents at damping ratios other than five percent by examining its response spectral values for low (two percent), and high (ten percent) damping ratios.

f. For any RSC motion generated from a starting acceleration time history containing a "fling". The corresponding velocity and displacement time histories shall be checked against the velocity and displacement time histories of the starting motions, in order to assure that the fitting process did not substantially alter the characteristics and amplitude of the "fling".

g. For each generated three-component set of RSC histories, the cross-correlation coefficient for each pair of acceleration, integrated velocity or integrated displacement histories shall be computed and verified to have a relatively low value (say 0.1 or less for acceleration, 0.2 for velocity, and 0.3 for displacement histories).

9.4.4 Digitization Interval

A suitable digitization interval must be specified before using an acceleration time history for analysis purposes. The maximum time step used in time-history dynamic analysis shall be between 1/6 and 1/10 of the lowest significant period of the headworks structure considered.

For equivalent-linear (EQL) analyses of most embankment dams, a digitization interval of 0.02 second is sufficient. Infrequently, a digitization interval of 0.01 second will be required to properly simulate EQL dynamic response. However, some nonlinear embankment dam analyses require a very short time step for the calculations, sometimes as low as 10^{-4} to 10^{-5} second, in order not to miss some possibly essential irreversible effects. In that case, one simply interpolates linearly between the digitized values of the specified input motion to re-define the seismic input at more closely spaced intervals.

In all cases of nonlinear studies, the suitability of the integration time step (if kept constant throughout the entire analyses) shall be verified through a sensitivity evaluation of the results obtained. Such verification is essential in the case of nonlinear computer programs that do not automatically verify the largest allowable time step, at any given time in the calculations, to ensure numerical stability and a correct simulation process. However, in order to save on the overall required computation time, it may be appropriate to adjust the analysis time step during various phases of a nonlinear response evaluation. A larger analysis time step may be acceptable under low to moderate levels of input motion, while a shorter time step is required during the strongest phase of shaking or whenever nonlinear response is occurring.

Concrete dams respond at higher frequencies than embankment dams, and a digitization interval of 0.01 second has become the standard for linear-elastic response analyses (as is the most frequently used). Some concrete dams may require a shorter digitization interval. For example, multiple arch dams often need consideration of numerous modes of vibration, in order to obtain adequate mass participation in the response analyses. In such cases, and to properly excite the higher modes of vibration, it may be appropriate to shorten the analysis time step to 0.005 second or less.

Similar to nonlinear analysis of embankment dams, nonlinear analysis of concrete dams requires very short digitization intervals. The presence of very stiff solid elements in the numerical model of a concrete dam also contributes to the need for a very short time step in explicit nonlinear analyses.

9.5 Location of Application of Input Motions

Response spectra or acceleration time histories are typically developed for free-field condition and, therefore, represent ground motions expected at the site ground surface, in the absence of the dam. Frequently, seismic input is directly applied at the base of the dam model, if the dam is founded on quasi-rigid bedrock, or at the base of the foundation model, if the dam is founded on alluvium or flexible rock.

Applying free-field input to the base of a numerical model may lead to response inaccuracies, although such inaccuracies will generally err on the conservative side.

If a deep alluvium or weak soils are present, one generally needs to enter the input motion at the base of a numerical model of the headworks structures and foundations, at bedrock level, or at some suitable depth within the foundation materials. In order to obtain realistic input excitation, one may need to modify "free-field" seismic criteria to represent shaking “within" the profile represented. One may need to deconvolve input motion specified as "outcropping" to the bottom of the foundation layers included in the numerical model. This may be done by conducting preliminary one- or two dimensional site response studies (without the dam being represented). Considerable care must be exercised in the deconvolution process to obtain plausible results and avoid unrealistic frequency characteristics in the computed base motion.

9.6 Probabilistic Seismic Hazard Evaluation

A probabilistic seismic hazard analysis (PSHA) involves relating through mathematical and statistical processes a ground motion parameter and its probability of exceedance at the site, for a specified duration of time (such as the operating life of the dam). The value of the ground motion parameter to be used for the seismic evaluation is then selected after defining a probability level, applicable to the headworks structures and site considered, and acceptable to the owner, regulatory authorities and other concerned parties.

Recognized active or capable faults and/or seismic source zones shall be considered as seismic sources in the PSHA model. The geographic relationship between the headworks site and seismic source(s) of concern to the site and the rates of activity assigned to each source form the basic elements of the model. Such models shall be consistent with the local and regional geologic and tectonic settings and with the applicable historic and geologic rates of seismic activity. The evaluation of the seismic hazard at a headworks site due to a single source involves convoluting three probability functions:

a. The probability that an earthquake of a particular magnitude will occur on this source during a specified time interval.
b. The probability that the rupture associated with this source and that magnitude event will occur at a specified distance from site.
c. The probability at the ground motion from an earthquake of those magnitude and distance will exceed a specified level at the site.

By combining for each source these three probability functions and by adding up the contributions from all sources, one can compute the probability of exceeding a specified level of ground motion at a given site, for any specified interval of time.

The probabilistic analysis can be used for PGA or PGV, as well as for spectral amplitudes at successively specified periods, thereby producing “uniform-risk” or “consistent” response spectra. Generally, at least two earthquakes, a near-field and a far-field earthquake will be needed to approximate the uniform-risk spectrum. The main advantages of a PSHA, over the deterministic approach, are the consideration of the design life of the project and the ability to select a level of risk acceptable to all parties involved. Other benefits include the following:

a. Inclusion of the contributions from all significant earthquakes, ranging from the smallest magnitude of concern up to the maximum magnitude on each source.
b. Inclusion of the contributions from all sources and all distances.
c. Selection of seismic parameters that correspond to acceptable levels of risk.
d. Avoidance of unnecessary conservatism.

For dam projects, an annual exceedance probability between 1/3,000 and 1/10,000 (3,000 to 10,000 year return period) is recommended to define input motion representing the MDE, depending on the applicable risk rating.

PSHA can be also used to complement a deterministic analysis. The probabilistic analysis aids in deciding whether to select mean or mean-plus-one-standard-deviation estimates of the ground motion, consistent with an acceptable probability of exceedance. PSHA provides an invaluable aid to judgment in defining the MDE. While a deterministic approach is easier to justify, at least in the case of the MDE, the probabilistic approach offers more consistency in the way it can be applied to various dams and sites of comparable risk ratings. In some cases probabilistic criteria may be more conservative than deterministic criteria. This is-either because of the way PSHA accounts for the uncertainty in the attenuation equations (error term), or because PSHA assumes hypothetical event locations (such as those associated with the background seismicity) that may be centered closer to the dam than applicable active fault traces. In such case, a deterministic review shall be used to temper extreme probabilistic estimates, after careful consideration of the locations and rates of activity of the concerned seismic sources.

PSHA will always provide useful insight to the engineer to better assess the actual risk associated with a deterministic MDE. The engineer shall avoid excessively conservative procedures be used either concurrently or at successive steps of a dam safety evaluation. Hence, use of the deterministic and probabilistic methods, whenever possible, for the case of the MDE is recommended. The probabilistic approach is preferable to specify the OBE.

9.7 Selecting the Controlling Earthquakes

9.7.1 Maximum Credible Earthquake (MCE)

The first step for selecting the controlling MCE is to specify the magnitude and/or modified Mercalli (MM) intensity of the MCE for each seismotectonic structure or source area within the region examined around the site. The second step is to select the Controlling MCE based on the most severe vibratory ground motion within the predominant frequency range of the dam and determine the foundation dislocation, if any, capable of being produced at the headworks site by the candidate MCE's. If more than one candidate MCE produce the largest ground motions in different frequency bands significant to the response of the dam, each shall be considered a Controlling MCE (CMCE). Inelastic behavior with associated damage is permissible under the MCE.

9.7.2 Maximum Design Earthquake (MDE)

In the event of the MDE the associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of a reservoir that would result into loss of life or significant damage to property and environment.

The MDE can be characterized as a deterministic or probabilistic event:

a. For critical structures the MDE shall be set equal to the MCE. Critical structures are defined as structures of high downstream hazard whose failure during or immediately following an earthquake could result in loss of life.
b. For other than critical structures the MDE shall be selected as a lesser earthquake than the MCE, which provides for an economical design meeting specified safety standards. This lesser earthquake shall be chosen based upon an appropriate probability of exceedance of ground motions during the design life of the structure (also characterized as a return period for ground motion exceedance).

9.7.3 Operating Basis Earthquake (OBE)

The selection of the OBE shall be based upon the desired level of protection for the project from earthquake induced damage and loss of service project life. The project life of new dams is usually taken as 100 years. The probability of exceedance of the OBE during the project life shall be no greater than 50 percent unless the cost savings in designing for a less severe earthquake outweighs the risk of incurring the cost of repairs and loss of service because of a more severe earthquake. This corresponds to a return period of 144 years for a project with a service life of 100 years. Headworks structures should be capable of resisting the controlling OBE within the elastic range, remain operational, and not require extensive repairs.

In a site-specific study the OBE is determined by a PSHA, which involves:

a. Developing a magnitude frequency or epicentral intensity frequency (recurrence) relationship of each seismic source.

b. Projecting the recurrence information from regional and past data into forecasts concerning future occurrence; attenuating the severity parameter, usually either PGA of MM intensity, to the site.

c. Determining the controlling recurrence relationship for the headworks site; and finally, selecting the design level of earthquake based upon the probability of exceedance and the project life.

10. INTERDISCIPLINARY COORDINATION

Earthquake resistant design of headworks structures requires a team of engineering geologists, seismologists, and structural engineers. They shall work together in an integrated approach so that elements of conservatism are not unduly compounded. An example of undue conservatism includes using a rare event as the MCE, upper bound values for the PGA, upper bound values for the design response spectra, and conservative criteria for determining the earthquake resistance of the structure. The steps in performing design shall be fully coordinated to develop a reasonably conservative design with respect to the associated risks. The structural engineers responsible for the structural aspects of the diversion structures shall be actively involved in the process of characterizing the earthquake ground motions in the form required for the method of analysis to be used.

The design objectives and its possible modes of failure control the analysis requirements to a large extent, and, therefore, the way in which seismic parameters shall be selected and specified. Various methods of analysis call for different ways of specifying earthquake motions, for a given earthquake level. Therefore, effective communications shall be established between the geologists, seismologists and engineers responsible for specifying the earthquake motions, and the engineers who will analyze the diversion structures for those motions.

PART 2A - HEADWORKS PLANNING AND LAYOUT

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Part

2A

Headworks Planning and Layout

1. PURPOSE

Part 2A of the *Design Guidelines for Headworks of Hydropower Projects* provides guidance on the overall planning and layout of the headworks of run-of-river hydropower projects in Nepal. The guidelines presented herein are intended to ensure that the technical, economic, safety and environmental factors which influence the development and operation of such headworks are adequately considered in their planning and layout. They are also aimed at assisting the headworks designer comprehend the planning issues that stem from conditions typical to Nepal and thereby incorporate them suitably into the planning and layout of the headworks.

In recognition of the fact that each headworks arrangement is site specific, the guidelines do not attempt to set forth standardized planning and layout criteria for the headworks. It is anticipated that the agency responsible for the development of the headworks will adopt suitable measures in their planning and layout to satisfy the principles and requirements defined herein.

2. SCOPE

The guidelines focus on the general principles for the planning and layout of headworks of simple and pondage run-of-river hydropower projects. They discuss the headworks planning considerations, typical headworks components and their general arrangement, site selection considerations and factors influencing the selection of major headworks structures.

Wherever necessary, the guidelines highlight typical Nepali conditions that demand special attention in the headworks planning and layout and suggest suitable measures for addressing the requirements emanating from these conditions. For this purpose, the guidelines integrate the knowledge and experience gained from existing run-of-river hydropower plants in Nepal with general planning principles.

3. TERMINOLOGY

Terms used in these guidelines are defined below:

Barrage	Barrier with low crest provided with a series of gates across the river to regulate water surface level and pattern of flow upstream and other purposes, distinguished from a weir in that it is gated over its entire length and may or may not have a raised sill.
Concrete gravity dam	A solid concrete dam so designed and shaped that its weight is sufficient to ensure stability against the effects of all imposed forces.
Settling basin	Structure provided to remove suspended sediments from the water abstracted from the river.
Earthfill dam	Dam with the main section composed of gravel, sand, silt and clay.
Embankment	A dam constructed of excavated natural materials.
Headworks	All structural components of a hydropower project required to abstract water from the river to the waterways of the project.
Intake	Structure in the headworks where the water to the power plant is abstracted or separated from the river flow.
Non-overflow section	Section of the river diversion structure that is designed not to be overtopped.
Overflow section	Portion of a river diversion structure, usually occupied by a spillway, which allows the overflow of water downstream. Also referred to as spillway section.
Pondage run-of-river plant	Hydropower plant with sufficient storage to permit daily regulation of river flows during low flow seasons.
Rockfill dam	Dam composed of rock as a major structural element and having a core wall in the form of an impervious membrane placed either within the embankment or on its upstream slope.
River diversion structure	Fixed structure built for the purpose of diverting part or all of the water from a river out of and away from its course.
Simple run-of-river plant	Hydropower plant which generates power using the flow of the river as it occurs.
Spillway	Structure over or through which flood flows are discharged downstream.
Weir	Barrier across a water course to raise the water level in the course with or without low gates.

4. RUN-OF-RIVER HYDROPOWER PROJECTS

Run-of-river hydropower projects may be developed as a simple run-of-river scheme or a pondage run-of-river scheme†. For a particular site, the selection of the type of scheme shall be made during the overall project planning to attain optimum utilization of the hydropower potential of the site.

4.1 Simple Run-of-River Hydropower Projects

Simple run-of-river (SRoR) hydropower projects shall be designed to generate power using the flow in the river as it occurs. Due to the large seasonal variations in flow in Nepali rivers, these projects shall normally be designed to utilize the flows in the river during the

† Also refered to as peaking run-of-river scheme

dry season. The installed capacity of these schemes shall be based on the dependable flow available in the river throughout the year, with deductions for mandatory flow releases for downstream water needs.

4.2 Pondage Run-of-River Hydropower Projects

Pondage run-of-river (PRoR) hydropower projects shall be designed to generate power according to fluctuations in the power demand through hourly regulation of the daily flow available in the river. This capability shall be achieved through daily pondage of the water at the headworks during dry seasons when the flow in the river is less than the design discharge. During the rainy season when the river discharge exceeds the design discharge, the plant shall operate like a SRoR plant to facilitate bed load flushing.

In order to provide peak load coverage, the installed capacity of PRoR schemes shall be sized for flows larger than the dependable river flow. Based on site conditions and optimization, peak requirements shall usually be fixed for about 4 to 6 hours of plant operation during the dry season.

4.3 Choice between Schemes

Although both types of projects generate the same amount of daily energy under a given set of conditions, the choice between the two schemes shall be made considering the following factors:

a. Owing to their greater flexibility in operation, the PRoR projects are capable of bridging the gap between water availability and power demand and are, as such, more suited to the power system in Nepal.
b. The additional cost of storage facilities required for PRoR projects makes their energy costlier than that generated from SRoR projects.

5. OBJECTIVES OF PLANNING AND LAYOUT

The planning and layout of the headworks of run-of-river hydropower projects shall aim at attaining an optimal structural arrangement of the headworks that satisfies its functional requirements in a safe, reliable, economical and environmentally sustainable manner.

6. SCOPE OF PLANNING AND LAYOUT

The objectives stated in Section 5 shall be achieved through a techno-economic evaluation of different alternatives of the headworks with due thought to their environmental impacts. The alternatives may lie in the sites for the headworks structures, the types of headworks structures and the general arrangement of these structures. Accordingly, the planning and layout shall consist of the following activities:

a. Site selection for the headworks.
b. Selection of appropriate headworks structures.
c. General arrangement of selected structures.

These activities shall be performed in accordance with the general principles and planning considerations discussed in the following sections.

7. FUNCTIONAL REQUIREMENTS OF HEADWORKS

The headworks of run-of-river hydropower projects shall be planned and designed to ensure safe and regular power generation from the hydropower plant under normal conditions. For this purpose, the headworks shall fulfill the following functional requirements:

a. Withdrawal of desired quantity of water from the river for power generation.
b. Safe passage of flood flows.

c. Passage of trash, floating debris and ice.
d. Passage of sediments.
e. Bed control at the intake.
f. Exclusion of suspended sediments.
g. Flushing of settled sediments.

7.1 Withdrawal of Water

At all times of the year, the headworks of run-of-river plants shall be capable of withdrawing the desired amount of water from the river for power generation and allowing the surplus water, if any, to flow through. Accordingly, the headworks of simple run-of-river plants shall be able to extract the plant's design discharge from the river even during the dry season. Likewise, the headworks of pondage run-of-river projects shall be able to draw the required quantity of water, limited to the plant's design discharge, from the river in the dry season by hourly regulation of the daily pondage created at the headworks.

7.2 Passage of Floods

The headworks shall permit safe passage of design floods without any serious damage to its structures. Considering the uncertainties in flood estimates due to the limited flow records available for most Nepali rivers, the headworks arrangement shall also be sufficiently flexible to pass floods larger than the design flood with minimal structural damage. In addition, the headworks shall be planned to handle, with some structural damage, flash floods due to natural hazards such as Glacier Lake Outburst Flood (GLOF), Cloud Outburst Flood (CLOF) and overtopping or breach of temporary dams created by landslides along the river course.

7.3 Passage of Trash, Floating Debris and Ice

The headworks arrangement shall facilitate passage of the large amount of trash and floating debris, including timber and vegetation, brought along by Nepali rivers during the monsoon season. Headworks constructed at high elevations in Nepal shall also permit passage of ice during the winter season. To prevent clogging of the river intake, the headworks shall also have suitable provisions for cleaning of the trash and floating debris.

7.4 Passage of Sediments

The headworks arrangement shall facilitate safe passage of the heavy sediment loads, both suspended load and bed load, carried by Nepali rivers during the monsoon season (June to September). In particular, it shall ensure that the main bulk of the bed load, consisting of large boulders of several tonnes, is passed downstream of the headworks without causing significant structural damage or distress to the headworks components. To this end, it shall also prevent this bed load from approaching the intake during floods. For PRoR projects, arrangements shall also be made for sediment flushing during monsoon so that the pondage volume is not reduced due to silt deposition.

7.5 Bed Control at Intake

Under all conditions of flow, the headworks arrangement shall prevent build up of the river bed in front of the intake due to deposition of sediments brought along by the river. This requirement shall be complied with to ensure that the deposits do not block the intake area, cause uneven flows over the intake or permit entry of the bed load past the intake.

7.6 Exclusion of Suspended Sediments and Air

The headworks shall ensure exclusion of as much of the suspended sediments as necessary from the extracted water to minimize abrasion damage to the penstocks and the turbines runners. In particular, it shall be capable of removing the sharp-edged fines sediments, such as quartz, often carried by Nepali rivers especially during the monsoon season. If there is a possibility of pressure flow, the headworks shall be able to effectively remove air bubbles entrained in the extracted water.

7.7 Flushing of Settled Sediments

The headworks arrangement shall be able to efficiently flush out the sediments settled in the settling basin to ensure that its settling capacity remains unaffected. Preferably, the flushing operations shall not result in power outages.

8. TYPICAL HEADWORKS COMPONENTS

In order to satisfy the functional requirements in Section 7, the headworks of run-of-river hydropower projects shall typically consist of the following components:

a. A river diversion structure to divert water to the power plant, with or without pondage.
b. A spillway to permit excess flows or floods to be passed safely over or around the diversion structure.
c. One or more outlets to permit controlled discharges to be made to the river.
d. An undersluice for bed control at the intake and prevent the inflow of bed load to it.
e. Energy dissipating structures to return the flow past the spillway to the river without causing serious scour or damage.
f. An intake to let the water into the water conveyance system under controlled conditions.
g. A settling basin, with an optional spillway, to exclude the suspended sediments from the water entering the hydraulic conveyance system and flush out the settled sediments.

In addition, the headworks may have other associated structures and facilities to supplement the above components. These structures and facilities could include the following:

a. An approach canal (conduit), with silt/gravel excluder and an optional bypass spillway, connecting the intake and the settling basin.
b. Upstream divide walls to facilitate flow in front of the intake.
c. Downstream divide walls to ensure jump formation in overflow bays and to prevent scouring near the diversion structure.
d. River training works to minimize cross flows and permit axial river flow through the diversion structure and to provide favorable curvature of flow at the intake.
e. Fish ladders, fish bypass systems, etc. to mitigate environmental effects.
f. Hydro-mechanical equipment, such as gates, stop logs, trash racks, hoisting mechanisms, trash rake, etc.
g. Access to and within the headworks area.
h. Communication facilities.

9. DATA REQUIREMENTS

The planning and layout of headworks shall be carried out based on data on the topography, geology, hydrology, meteorology, sedimentology and environment of the headworks area. The principal data requirements for these aspects of the headworks area are listed below. These data shall be obtained through the survey and investigation procedures presented in the respective sections of Part 1 of the guidelines.

Topography

a. General topography of the area around the headworks site, including its catchment.
b. Natural and manmade features in the vicinity of the headworks, including major infrastructure, settlements, etc.

c. Detailed topography of the proposed headworks area, including major landforms, elevations, etc.
d. Longitudinal profile and cross-sections of the river at, upstream of and downstream of the proposed site for the diversion structure.

Geology

a. Regional and local geological structures.
b. Site-specific surface and subsurface geology of the headworks area, including lithology and physical conditions of various strata.
c. Hydrogeology of the headworks area.
d. Engineering and index properties of the foundation materials.
e. Seismo-tectonic characteristics of the headworks area.
f. Locations of construction material sources, including the quality of materials and their reserve estimates.

Hydrology, Meteorology and Sedimentology

a. Maximum flood discharge and high flood level.
b. GLOF potential and discharges at the site.
c. River stage and discharge at or near the proposed site, including those during floods.
d. Discharge data at existing downstream or upstream gauging stations.
e. Rainfall data of rainfall gauging stations in and around the catchment.
f. Estimate of the quantity of suspended sediments and bed loads in the river.

Environment

a. Settlements, agricultural lands, cultural, religious or archaeological sites, etc. around the headworks area.
b. Downstream water uses such as irrigation, water mills, etc.
c. Terrestrial flora and fauna, including endangered and protected species, habitats, bio-diversity, etc.
d. Aquatic flora and fauna, including types of migrating fish in the river.
e. Navigability of the river before and after the construction of diversion structure.
f. Demographic characteristics, cultural resources and economy of the headworks area.

10. SELECTION OF HEADWORKS SITE

The headworks site shall be selected based on its suitability for the principal components forming the headworks scheme. For this purpose, alternative layouts of the headworks shall be prepared and compared based on careful consideration of technical, economic and environmental factors, and the site offering the techno-economically most feasible layout, with least environmental impacts, shall be selected for the headworks. The primary factors that shall be considered in the site selection are discussed in the following sections.

10.1 Topography

The diversion structure shall, as far as possible, be sited in a straight reach of the river so that uniform velocities and a fairly constant river cross-section are available at the structure. The river banks in this stretch shall be high, well-defined and inerodible to obviate oblique approach and non-uniform distribution of flow on to the diversion structure.

To minimize the volume of construction materials, the diversion structure shall be located at a narrow section of the river. However, a very narrow river width shall be avoided to ensure that the economy in construction materials achieved at such sites is not offset by the extra cost of deep cutoffs, deep foundations and elaborate surface protection works necessitated by high intensity flows resulting from the reduction of the waterway at such sections. Very narrow river widths shall also be avoided to guarantee that the hydraulic functions of the headworks components do not interfere with each other.

In rivers with potential for GLOF, a river section wider than that desired for economy may be adopted for the diversion structure. This shall be done to permit lower flood water levels at the diversion structure during GLOF.

Notwithstanding the desirability for a straight river reach for the diversion structure, a slight curvature in the river course shall be preferred for locating the intake to limit the amount of sediment drawn into it. The headworks area shall also have ample space for constructing the settling basin and sufficient level differences to allow flushing of the sediments deposited in the basin.

10.2 Foundation and Site Geology

As far as possible, the diversion structure shall be founded on sound bedrock with favorable geological characteristics and good engineering properties. Where bedrock is not available at reasonable depths, the diversion structure may be founded on alluvial river beds, subject to restrictions on its type or size. If deficient or dangerous geologic behavior is suspected, the foundation shall be modified or treated or, in the worst case, abandoned.

For successfully dissipating the energy of excess flows, spillways shall be sited on the most erosion-resistant foundation material present found along the axis of the diversion structure, preferably hard rock. As river beds are naturally more erosion resistant than the flood plains, gates and high velocity outlets shall be located in river beds rather than on flood plains to reduce the downstream erosion that may be caused by the new structure.

For economy and ease in construction, the diversion structure, spillway, intake and settling basin shall be located in areas that require minimal foundation treatment. Sites with active faults shall normally be avoided. As far as possible, sites with homogeneous lithology shall be chosen for the headworks structures. The selected sites shall also be geologically stable so that they do not undergo mass sliding due to fluctuations in water levels or earthquakes.

10.3 Potential Hazard

Preferably, the headworks shall be located at sites where the upstream and downstream hazard potentials resulting from its construction and operation are minimal. Such hazards potentials could include activation of geological instabilities and seismic events, loss of life and property due to failure of the diversion structure, etc.

10.4 Availability of Construction Material

The headworks shall preferably be located at sites where the required construction materials, particularly those needed in large quantities, can be obtained from structural excavations or from borrow areas or quarries lying within easy haulage distance to the site. For economy, sources for sand and gravel shall normally lie within 20 km of the site while those for rocks shall fall within 10 to 15 km of it.

10.5 Diversion during Construction

The headworks site shall be selected considering river diversion and flood handling during the construction period. The site shall permit convenient diversion of the river around or through the proposed diversion structure without incurring excessive costs or affecting the scheduling of construction activities. It shall also minimize serious potential flood damage to the construction works without the need of expensive protection works.

10.6 Construction Requirements

The headworks site shall have sufficient and suitable spaces available for facilities needed for construction of the headworks. These facilities shall include the contractors' lay down areas, storage, workshops, aggregate processing plant, batching and mixing plants, camps, etc. In addition, easy and unimpeded access within the site shall be available throughout the year. for safe and economical transportation of equipment and materials.

10.7 Infrastructure

As far as possible, the headworks shall be located at or near sites with existing infrastructure such as access road and power supply. Sites that require construction of long access roads or transmission lines for power supply shall generally be avoided.

10.8 Environmental Considerations

The headworks site shall be selected with due consideration to minimizing adverse impacts of the headworks construction or operation on the physical, biological, socio-economic and cultural environments of the area. Some of the primary environmental concerns that shall be addressed during site selection are:

a. Interference of headworks with existing habitation, infrastructure, agricultural lands, etc.
b. Inundation resulting from formation of pond.
c. Activation of geological instabilities and seismic events.
d. Spoil and waste disposal.
e. Loss of vegetation due to placement of headworks structure and facilities.
f. Loss and change of aquatic habitat due to river flow reduction, pondage formation, etc.
g. Loss or fragmentation of habitat and bio-diversity.
h. Effect on downstream usage of water.
i. Encroachment of historical and cultural sites, tourist areas, etc.
j. Effect on ethnic groups, tribal groups, tourist areas, etc.

11. GENERAL ARRANGEMENT

The general arrangement of the headworks shall be site specific. However, the arrangement shall follow the principles discussed below to satisfy the functional requirements discussed in Section 7.

11.1 Diversion Structure

The diversion structure shall generally be planned on a straight axis, but it may also be slightly curved if the upstream curvature locates that part of the diversion structure on higher bedrock foundation and thereby adds to economy and safety. As far as possible, the structure shall be aligned perpendicular to the river course to minimize its length and to ensure normal and uniform flow through its bays. Unless required by site conditions, skew alignment of the structure shall be avoided to minimize chances of shoal formation and shrouding portions of the structure, especially the undersluice pocket.

The overall length of the diversion structure shall be fixed based on the site topography and hydraulic requirements. Its crest level shall be determined considering the head needed to pass the available flow, less the environmental release, during the dry season. Likewise, the profile of the diversion structure shall be chosen based on the site conditions and the quantity of water to be handled.

11.2 Spillway

The spillway may be provided as an integral part of the diversion structure or as a separate structure. In concrete diversion structures, the spillway shall generally be located centrally, away from both abutments where erosion damage needs to be avoided. In embankments, the spillway shall be located on one side of the embankment at an erosion-resistant site. It shall not be straddled across the embankment to prevent the latter's failure by erosion at its foot or below the concrete chute that passes the flood.

The spillway arrangement shall be decided considering the location, type and size of other components of the diversion structure, including the stilling basin or the energy dissipation device provided downstream. However, it shall be located close to the intake to enable passage of trash and debris past the intake and to flush out bed load deposits in front of it.

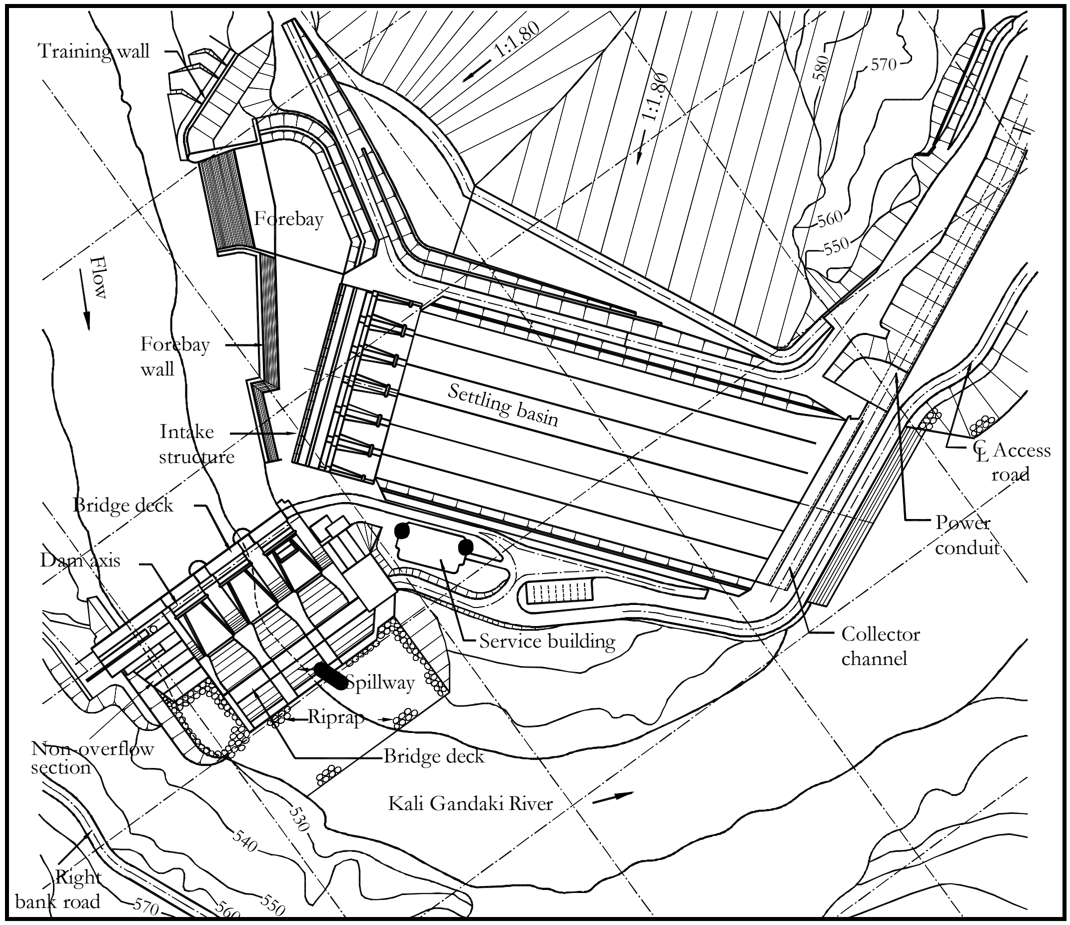

Figure 1: Headworks of Kali Gandaki "A" Hydroelectric Project, Nepal (Nepal Electricity Authority 2002)

11.3 Intake

To minimize sediment entry into the water conveyance system, the intake shall normally be aligned at an angle of 90º to 110º to the axis of the diversion structure. A bed-load sluice may be located below the intake to separate out smaller stones and gravel which may follow the abstracted flow.

In sand and gravel-bed rivers, the intake shall preferably be located at the downstream end of the convex bend of the river so that the secondary currents generated at these locations can be used to advantage in reducing the influx of sediments to the intake. While selecting this location, care shall be taken to protect the intake foundation against cross currents that are likely to be produced at such locations. Adequate care shall also be taken to protect the intake from floating debris attracted along with the surface water at such locations.

In steep rivers lined with boulders and rock outcrops, however, the intake shall not be sited at convex bends in the river to prevent the intake and its foundation against damage from the hydraulic loads and the impact from boulders transported by the river during floods. The intake may be located in a more protected area even if more sand and gravel will follow the water towards the intake.

11.4 Undersluice

The undersluice shall be provided close to the intake to flush out the sediments deposited in front of the intake and thus control the bed levels in its approach area. The opening of the undersluice shall be sized to pass the largest possible boulders brought along by the river. Based on the flow to be handled, the undersluice may consist of one or more bays; however, for small rivers flows, a single sluice bay may be preferred over two bays with smaller openings.

The profile of the undersluice shall mainly be based on the discharge to be passed through it. Its crest and upstream floor levels shall generally be kept at the lowest bed level of the deep channel of the river, subject to the cost of foundation and the difficulty in dewatering.

11.5 Gravel Trap

In certain cases, a gravel trap may be provided in front of the intake or below its invert level. It shall be proportioned based on the debris content and the size of the gravels present in the river water.

11.6 Settling Basin

Depending on the availability of sufficient space and flushing head, the settling basin may be integrated with the intake or be connected to it through an approach channel/conduit. It shall be provided with flushing channels and control arrangements for sediment sluicing. It may also be provided with side or end spillways to spill the excess water in it back to the river.

Depending on site conditions and project requirements, the settling basin may be a single-chamber or a multi-chamber basin. Considering the heavy silt conditions in Nepali rivers, a minimum of two chambers shall be considered.

11.7 Divide Walls

A divide wall shall normally be constructed to separate the undersluice bays from the other bays of the diversion structure. Under adverse flow conditions, additional divide walls may be required in the diversion structure.

Divide walls shall be positioned at right angles to the axis of the diversion structure. On the upstream side, the divide wall shall extend from two-third to the full width of the diversion structure to obtain a pocket of comparatively still flow at the intake for sediment deposition. On the downstream side, the wall shall generally extend to the end of impervious floor or to the end of loose apron on the downstream side to ensure adequate tail water depth for jump formation and to prevent cross flows that could cause objectionable scours.

11.8 Fish Pass Structures

Fish ladders shall be located in areas with high flow of water. To increase their effectiveness, attraction flows or leaders shall be arranged to guide the fish to the fish ladder or the fish bypass system.

12. SELECTION OF RIVER DIVERSION STRUCTURE

For a particular project, the type of river diversion structure shall be selected from amongst a range of such structures deemed suitable for headworks of run-of-river projects, especially under construction, operation and maintenance conditions prevalent in Nepal. The choice of the most suitable diversion structure shall be made in accordance with the considerations discussed in the following sections.

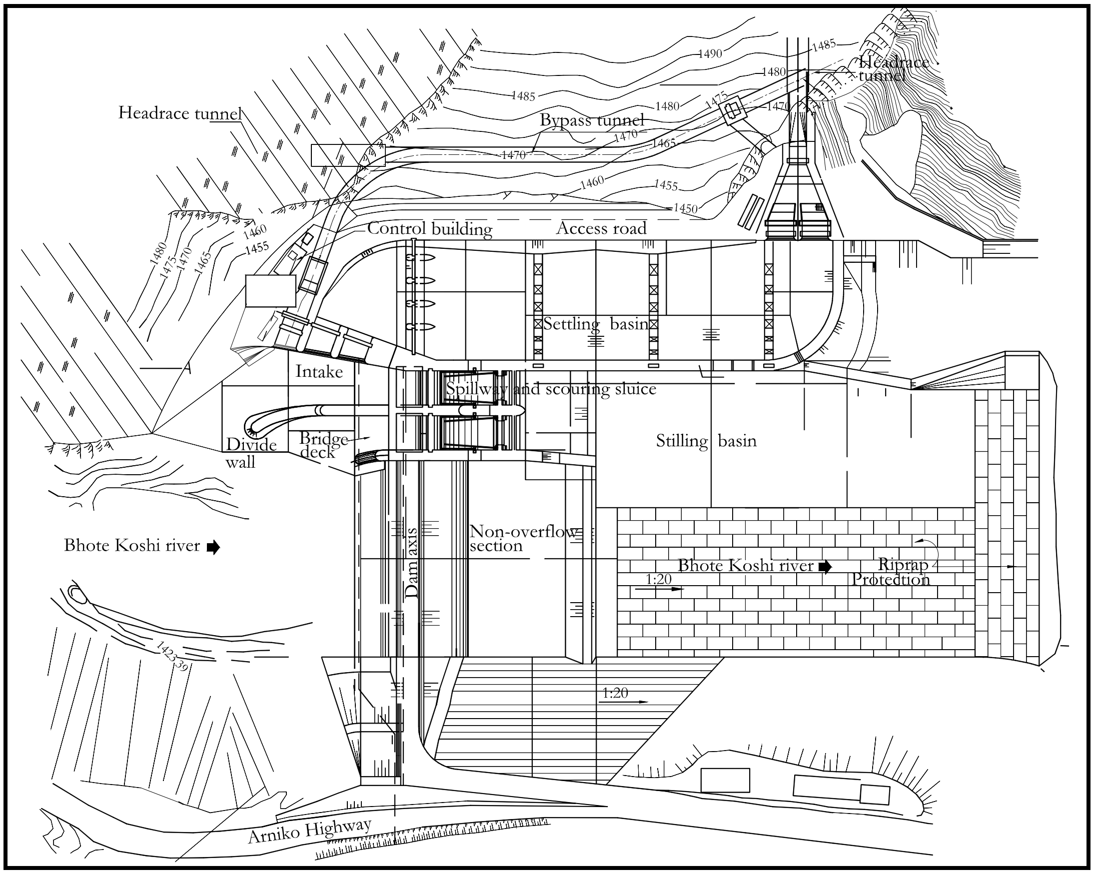

Figure 2: Headworks of Upper Bhote Koshi Hydroelectric Project, Nepal (courtesy: Bhote Koshi Power Company)

12.1 Types of River Diversion Structures

The river diversion structure for a run-of-river hydropower project may be provided in one of the following forms:

a. An overflow structure across the river with or without low gates, functioning as a weir.

b. An overflow structure across the river with gates on its crest, functioning as a barrage or a total spillway dam.

c. A non-overflow dam across the river with a separate gated overflow structure (spillway).

d. A composite structure comprising a non-overflow section and a gated overflow section (spillway).

Except for low diversion structures, the overflow structures shall be built with materials like concrete or masonry that are not eroded by the high discharges, large boulders, trash and debris passing over their crests. The non-overflow structure may be constructed in concrete, rockfill or earthfill.

Where, according to site conditions, a permanent diversion structure is not required or desired, temporary river diversion using methods like boulder pitching, gabions with timber or steel cribs, etc. may be utilized to divert low flow during the dry season. However, the reliability of power supply shall be given due consideration while opting for such arrangements.

12.2 Considerations for Selection

The river diversion structure shall be selected based on considerations of the type of run-of-river scheme, physical conditions of the site, economy and the environment. Where these considerations are in conflict, the choice of the diversion structure shall be guided by those considerations that have the greatest bearing on the safe reliable performance of the structure.

12.2.1 Type of Scheme

The headworks of simple run-of-river schemes shall be planned to divert the required water for power generation and let the surplus water flow through the river. For this purpose, a low head river diversion structure, mostly uncontrolled, shall suffice. This requirement may be met through a permanent or a semi-permanent weir across the river. The permanent weir may be built in concrete or masonry.

For pondage run-of-river schemes, the headworks shall be planned to store the river water on a daily basis and divert the required water for power generation on an hourly basis in the low flow season. During the rainy season, the headworks shall also be able to let the high flows pass safely. To fulfill these needs, a gate-controlled diversion/pondage structure shall be provided across the river. Depending on site conditions, this structure may be a concrete barrage (gate-controlled), a concrete or embankment non-overflow dam with a separate gated overflow structure or a composite structure consisting of a concrete or embankment non-overflow section and a gated concrete overflow section.

12.2.2 Topography

The diversion structure shall conform to the natural topography of its site. A concrete overflow structure shall be preferred at sites where a narrow river flows between steep, rocky canyon walls. In low rolling terrain, an embankment in earthfill or rockfill may be adopted.

12.2.3 River Slope

To reduce risks and problems in headworks operation and maintenance, barrages and other gated structures shall generally be avoided in very steep rivers, i.e. rivers with 1:30 or higher slopes, that are capable of transporting large boulders weighing several tonnes during floods. In steep rivers with slope milder than 1:30, weirs with low-level gates may be provided with an ungated overflow section. The crest of the low level gates shall preferably be located at the initial riverbed level in order to bring the riverbed back to its original gradient and thus facilitate removal of deposits upstream of the weir during flushing.

12.2.4 Hydrological Hazards

For rivers whose flood flows cannot be reliably estimated owing to limited flow records, a diversion structure capable of handling unexpected high floods, i.e. floods larger than the design flood, shall be provided. In such situations, a long free overflow weir with a low unit flow shall be preferred over a barrage or a fully gated spillway with a high unit flow. Where composite diversion structures are provided in such rivers, a concrete non-overflow section shall be selected over an embankment to ensure minimum damage to the structures under occasional overtopping. These considerations shall also apply in the selection of diversion structures in rivers prone to hazard floods resulting from GLOF, CLOF or overtopping or breach of upstream temporary dams.

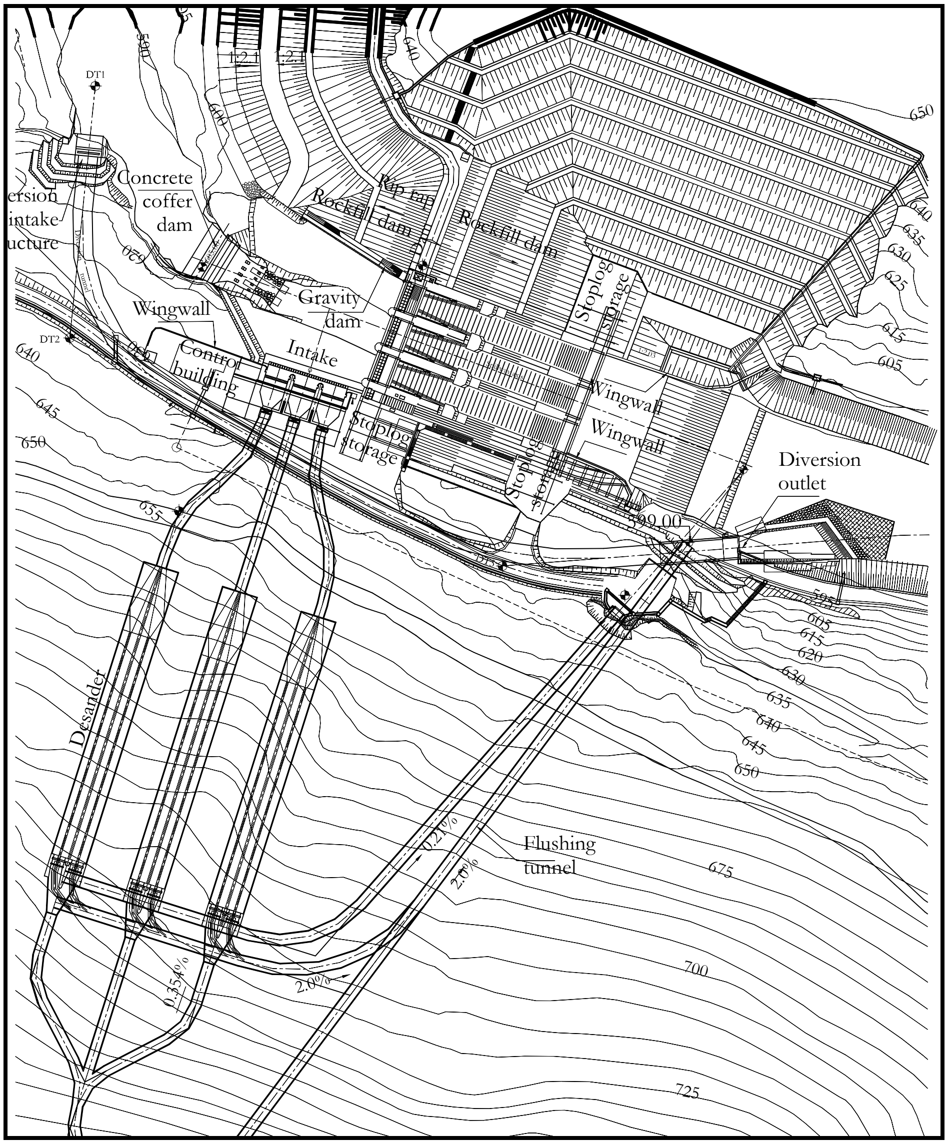

Figure 3: Headworks of Middle Marsyangdi Hydroelectric Project, Nepal (courtesy: NEA)

12.2.5 Sediment and Debris Content

Unless pondage is necessary for cost effectiveness of the hydropower project, barrages and gated overflow structures shall generally be avoided on rivers that carry large amounts of trash and debris during floods, including large trees during extreme floods. This measure shall be taken to safeguard against blockage of the waterways of gated structures by the trash and debris, resulting in unexpected rise in pondage levels even with the gates open. In such rivers, free overflow structures like weirs shall be preferred as they do not pose any threat of blockage or accumulation of trash.

12.2.6 Geology and Foundation Conditions

The choice of the diversion structure shall be limited by the foundation conditions, although such limitations may be modified considering the height of the proposed structure. Subject to economy of materials and overall cost, any type of diversion structure may be constructed on solid rock foundations. Low height weirs, barrages and embankments may be built on well compacted gravel foundations with due precautions to provide effective water cutoffs or seals.

12.2.7 Seismicity

Seismic considerations shall not limit the choice of the type of river diversion structure. However, the diversion alternatives chosen for a particular site shall be studied for their behavior under likely seismic events at the site, and alternatives that appear vulnerable to damage shall be eliminated from consideration unless such vulnerability can be taken care of by reshaping, resizing or other means.

12.2.8 Availability of Construction Materials

For economy in construction, preference shall be given to the diversion structure for which construction materials of required quality can be found in sufficient quantity from structural excavations or at reasonable haulage distances from the site. Concrete structures shall be chosen at sites where suitable coarse and fine aggregates for concrete are available at a reasonable cost locally and on property that is to be acquired for the project. However, if suitable soils or rocks are available at nearby borrow pits or quarries, an earthfill or a rockfill embankment shall be considered for the river diversion structure.

While applying this selection criterion, caution shall be exercised in eliminating an alternative due to unavailability of the required quality of material in sufficient quantity. At times, this constraint may be overcome by stretching small deposits of the required material by careful blending with materials of slightly inferior quality without compromising on the final quality of the structure.

12.2.9 Spillway Size and Location

The choice of the diversion structure shall frequently be controlled by the size and type of the spillway and the natural restrictions in its location. At sites where the run-off and river flow characteristics demand a large spillway, a concrete overflow structure in the form of a weir or barrage may be adopted. Small spillway requirements may favor the selection of earthfill or rockfill embankments even in narrow dam sites.

If site conditions permit or necessitate that the spillway be located away from the diversion structure, a non-overflow structure in concrete, earthfill or rockfill shall be provided for diversion. Where excavated material from such separate spillway is available in sufficient quantity, an earthfill embankment shall be preferred. However, if the spillway cannot be detached from the river diversion structure, a composite structure comprising a non-overflow section in concrete or embankment and a concrete overflow section shall be provided. If case the non-overflow section of the composite structure is an earth or rock embankment, suitable precautions against unequal settlements resulting from differential consolidation of the embankment and foundation shall be adopted in the design. Care shall also be exercised against cracking of concrete or opening of joints which could permit leakage from the channel into the fill, with consequent piping or washing away of the surrounding material.

12.2.10 Constructability

The diversion structure selected for a particular site shall be compatible with its accessibility and the local availability of labor and equipment. This factor may be particularly important in projects in remote locations or in projects which are on a tight completion schedule.

12.2.11 River Diversion during Construction

The mechanism for river diversion during construction shall be an important factor in the selection of the diversion structure. Where the diversion during construction can be attained through a channel around the site, an earthfill or rockfill embankment may be adopted for the non-overflow section of the diversion structure. If such diversion is not feasible because of large diversion flows, space constraints or high costs, a concrete river diversion structure shall be chosen so that the construction diversion can be passed over it without excessive damage to its partially or fully completed parts.

12.2.12 Operation and Maintenance

The choice of the diversion structure shall consider its expected performance probabilities and maintenance requirements. For this purpose, the operation and maintenance costs for alterative diversion structures shall be evaluated during the structure selection study, and the structure requiring least operation and maintenance costs without affecting the functionality or safety of the headworks shall be adopted.

As far as practicable, diversion structures that can be conveniently operated under the entire range of operating conditions, including hazard conditions, shall be opted for. At sites that are prone to uncertainties in flood flows or to hazard floods, structures that require minimal mechanical operations, such as weirs, shall be preferred so that mechanical failure or human errors in gate opening under flood conditions do not endanger structural or public safety. During the selection, care shall be taken to ensure that the selected structure does not pose costly and disruptive problems during its operation.

The selected diversion structure shall require minimal maintenance of hydraulic and hydro-mechanical components. The selection shall consider the potential sources of damage to the structure, such as damage to overflow surfaces and gates by boulders and debris, which arise from conditions typical to the site or the river. Structures that could force frequent outages of the hydropower plant for repair and maintenance shall be eliminated from consideration.

12.2.13 Environmental Considerations

The choice of a river diversion structure at a particular site shall be based on a comparison of the adverse environmental impacts of alternative diversion structures during construction and operation. The comparison shall be performed in terms of the magnitude, extent and duration of the impacts of these structures on the physical, biological, socio-economic and cultural environments of the area, and the structure that inflicts the least adverse impacts on the environment without compromising on functionality or safety shall be selected. Some of the major parameters that shall be considered in this comparison are:

- Construction techniques, schedule, equipment and material.
- Site access.
- Extent of quarrying and burrowing.
- Nature and volume of spoil and waste disposal.
- Increase in river turbidity.
- Change in water quality of the river.
- Loss or fragmentation of wildlife habitat.
- Disturbance to terrestrial and aquatic flora and fauna.
- Change in sediment deposition pattern, water pollution and flow reduction.
- Relocation and resettlement of affected households.
- Aesthetics and compatibility with natural settings.

12.2.14 Life Cycle Cost

If several acceptable alternatives for the river diversion exist, the ultimate factor determining the type of diversion structure shall be the life cycle cost of the alternatives. For this purpose, the alternatives shall be compared according to a life cycle costing approach by balancing their investment costs with their capitalized operation, maintenance and repair costs, and the structure that meets the criteria of minimum capitalized life cycle cost shall be selected.

13. CASE STUDIES

While selecting the headworks plan and layout for any project, headworks of run-of-river plants on the same or similar rivers in Nepal shall be studied to evaluate their performance vis-à-vis their planning and structural layout. In the study, the operational problems at the headworks and their causes shall be identified, and the lessons learnt from such case studies shall be suitably reflected in the planning and layout of the headworks under consideration.

14. HYDRAULIC MODELING

The proposed headworks arrangement may generally be finalized through hydraulic model studies. The studies shall test the ability of the arrangement to provide good hydraulics and to satisfy the functional requirements of the headworks. Any deficiencies detected in the arrangement shall be suitably corrected.

PART 2B – CONCRETE DIVERSION STRUCTURES

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Part

2B

Concrete Diversion Structures

1. PURPOSE

Part 2B of the *Design Guidelines for Headworks of Hydropower Projects* provides technical criteria and guidance for the design of concrete diversion structures for headworks of run-of-river hydropower projects in Nepal. The guidelines are intended to ensure safe and economical design of these structures with due consideration of relevant issues, particularly those arising from conditions typical to Nepal.

2. SCOPE

The guidelines cover the design of concrete diversion structures deemed suitable for run-of-river hydropower projects in Nepal. These structures include weirs, barrages and diversion structures with non-overflow and gated overflow sections.

The guidelines deal with conventional concrete gravity diversion structures only. This class of diversion structures has been focused on primarily because its performance under typical Nepali conditions is well established and understood through its extensive use in existing run-of-river hydropower projects in Nepal. Other types of concrete diversion structures, such as hollow concrete gravity diversion structures and concrete buttress diversion structures, have not been included as the specialized experience and technology needed for them is not available in Nepal, and their increased cost of formwork and reinforcement steel required is known to offset the savings in concrete offered by them. Likewise, roller-compacted concrete diversion structures have not been considered as the economy in construction time offered by their technology is not likely to significantly influence the overall construction period of run-of-river hydropower developments envisaged in Nepal. Semi-permanent or temporary type of diversion structures, which are also planned in some projects, are not included in the guidelines as they are site specific in nature; however, the general principles discussed in the guidelines may be followed for design of such structures.

The guidelines presented in this part cover the hydraulic and structural design of the non-overflow and overflow sections of concrete gravity diversion structures on permeable and impermeable foundations. They discuss design considerations and data, selection of design parameters, determination of layout and profiles, hydraulic design, load conditions, stability requirements, stress analysis and structural design of these sections. Design procedures for other elements related to these sections, such as aprons, energy dissipators, fish passage, etc., are presented in other parts of the guidelines.

3. TERMINOLOGY

Terms used in these guidelines are defined below:

Afflux	Difference in water level at any point upstream of diversion structure before and after its construction.
Apron	Horizontal impervious surface provided on upstream and/or downstream side of diversion structure to reduce uplift and seepage.
Block-out	Temporary recess provided in structure to facilitate proper embedment of steel fixtures or gates, trestles, etc., and which are concreted after their fixing.
Cavitation	A hydrodynamic process that occurs behind irregularities at zones of low pressure in flow saturated with vapor gas bubbles.
Concrete diversion structure	A solid concrete diversion structure designed and shaped such that its weight is sufficient to ensure stability against the effects of all imposed forces.
Contraction joint	Formed surface, usually vertical, in a weir or barrage to create a plane for the regulation of volumetric changes
Crest	Top of overflow section over which water flows downstream.
Cutoff	Impervious construction placed beneath a river diversion structure to protect the structure against scour and possible piping due to excessive exit gradients of the seepage flow below the foundations.
Divide wall	Wall usually constructed at right angles to the axis of the weir or barrage and generally extending beyond the main structure to separate the undersluices, river sluices and spillways into independent bays for facilitating regulation in flow.
Exit gradient	Slope (or gradient) of hydraulic grade line for subsoil seepage flow at the exit end of the structure where the seepage water comes out from subsoil.
Fetch	Straight line distance along the wind direction (along central radial of fetch) over open water on which the wind blows.
Filter	Graded materials used for design of hydraulic structures.
Freeboard	Vertical distance between a specified still water pool surface elevation and the top of the diversion structure, without camber.
Full supply level	Level of water immediately upstream of the weir or barrage required to facilitate withdrawal of design flow discharge into the power intake.
Gallery	Long, narrow passage inside a river diversion structure used for access, inspection, grouting or drilling of drain holes.
Glacis	Sloping portion of the dam upstream and downstream of the crest of the river diversion structure.
Hazard	Situation which creates the potential for adverse consequences such as loss of life, property damage, etc.

Hazard potential	Potential for loss of human life or property damage in the area downstream of the diversion structure in the event of its failure or incorrect operation.
Heel of structure	Location where the upstream face of the diversion structure intersects the foundation.
Inflow design flood	Flood flow above which the incremental increase in water surface elevation due to failure of a diversion structure is no longer considered to present an unacceptable threat to downstream life or property.
Minimum freeboard	Difference in elevation between the top of the diversion structure and the maximum reservoir water surface that would result from routing the inflow design flood through the reservoir.
Monolith	Section or block of the river diversion structure bounded by transverse contraction joints.
Non-overflow section	Section of the river diversion structure that is designed not to be overtopped.
Normal freeboad	Difference in elevation between the top of the diversion structure and the normal maximum pool elevation.
Overflow section	Portion of a weir or barrage, usually occupied by a spillway, which allows the overflow of water. Also referred to as spillway section.
Overflow section capacity	Design inflow flood which a diversion structure can safely pass.
Stop log	Fabricated concrete, structural steel or wooden unit utilized for temporary closure of any bay to facilitate repair of gates and other components of the bay.
Tailwater elevation	Elevation of the water surface downstream from a weir or barrage.
Toe of structure	Location where the downstream face of the diversion structure intersects the foundation.
Transverse joint	A joint normal to the longitudinal axis of the weir or barrage.
Uplift pressure	Upward water pressure in the pores of concrete or rock or along the base of the diversion structure.
Water stop	Thin sheet of metal, rubber, plastic, or other material placed across joints in concrete diversion structures to prevent seepage of water through the joint.
Weir	Hydraulic structures in rivers over which controlled flow of water takes place.

4. DESIGN OBJECTIVES

A concrete river diversion structure shall be designed to create a hydraulically efficient and structurally adequate arrangement that satisfies the following functional requirements in a safe and economical manner:

a. Withdrawal of desired quantity of water for power generation, with or without pondage.

b. Passage of surplus flows to the downstream river channel.

c. Safe passage of flood flows to the downstream river channel.

d. Passage of trash, floating debris and ice.

e. Passage of sediments, including large boulders.

5. SCOPE OF DESIGN

The objectives listed in Section 4 shall be attained through proper hydraulic and structural design of different components of the diversion structure. Generally, the design shall entail:

a. General arrangement of the diversion structure.
b. Determination of suitable profiles and sections.
c. Hydraulic design of components.
d. Structural design of components.

These activities shall be performed in accordance with the design principles and procedures discussed in the following sections.

6. DESIGN PHILOSOPHY

Concrete diversion structures for run-of-river hydropower projects shall be designed as solid concrete structures capable of:

a. Providing a tight and impervious barrier for diverting and/or retaining water for power generation.
b. Maintaining stability against design loads through their geometric shape and the mass and strength of the concrete.
c. Providing adequate waterway for safely passing flood flows to prevent overtopping and possible failure of the diversion structure.
d. Providing adequate head and waterway to flush trash, debris and sediment loads without causing significant structural damage or distress to the waterways.

7. TYPICAL ELEMENTS

A concrete diversion structure for run-of-river hydropower projects shall typically consist of the following components:

a. A non-overflow section for diverting and/or retaining water.
b. An overflow section for passing excess flows or flood flows, consisting of the following:
    i. A gated or ungated control structure for regulating flows in the overflow section.
    ii. A discharge channel (glacis) for conveying flow released from the control structure to the downstream river bed via the terminal structure.
    iii. A terminal structure for returning the discharge through the overflow section to the river without causing serious scour or damage.
    iv. Training walls for guiding the flow from the overflow section into the downstream river channel.
    v. Divide walls for separating different overflow bays of the diversion structure.
    vi. Piers for supporting bridges and different mechanisms for gate operation.
    vii. Crest gates for controlling flows past the diversion structure.
    viii. Bridge for access to pedestrian and vehicular traffic between non-overflow sections and for access or support for the operating machinery for crest gates.
c. An undersluice for flushing bed loads past the river intake.
d. Galleries, shafts and other openings for foundation grouting, drainage, inspection, gate operation, etc.
e. Upstream and/or downstream aprons to reduce uplift under diversion structures on permeable foundations.
f. Cutoffs and filters to reduce uplift under diversion structures on permeable foundations.

g. Fish ladders, fish bypass systems, etc. to facilitate fish movement past the diversion structure.
h. Guide banks.

8. GENERAL ARRANGEMENT

Concrete diversion structures shall generally be constructed on a straight axis, but they may also be slightly curved or angled to accommodate specific site conditions. Curved plans may be considered if the upstream curvature locates that part of the diversion structure on higher bedrock foundation and thereby adds to economy and safety.

The non-overflow and overflow sections shall form integral parts of the concrete diversion structure. For a particular site, the relative lengths of these sections shall depend on the total river waterway and the length of overflow section needed to pass the design floods safely past the diversion structure. The lengths shall be decided based on provisions of Sections 10.1 and 11.2.2. In the limiting case, the diversion structure may consist of an overflow section only.

The overflow section may be gated or ungated. Gated overflow sections shall be divided into a number of bays with the help of piers. The width of the bays shall primarily depend on the maximum width of gate available or desirable. Unless required for constructing vehicular access bridges across them, ungated overflow sections shall have a single bay.

Depending on requirements for temperature control, the diversion structure may be divided into monoliths that are separated by transverse contraction joints. Monoliths of relatively high diversion structure may be further divided into several blocks using longitudinal joints. These joints shall be located according to the provisions of Section 1.1.

Except in low diversion structures, a system of galleries, adits and shafts shall be provided within the body of the diversion structure to furnish means of access and space for drilling and grouting. In gated structures, chambers and access galleries shall also be provided for installation, operation and maintenance of accessories and utilities.

Access bridges across overflow sections shall be supported on piers. In the case of ungated overflow sections with no vehicular traffic, access across the overflow section may be provided by a small access bridge or by stair shafts and a gallery beneath the spillway crest.

9. DESIGN DATA

Concrete diversion structures shall be designed based on data on topography, foundation properties, seismological parameters, hydrology, meteorology, sediments, environment and structural materials.

9.1 Topographical Data

Data on the topography shall be adequate to prepare the layout and profiles of the different components of the diversion structure, to estimate the upstream submergence due to water impoundment and to assess the downstream hazard potential due to failure of the diversion structure. These data shall primarily consist of the following:

a. General topography of the area upstream and downstream of the headworks site.
b. Natural and manmade features in the vicinity of the headworks, including major infrastructure, settlements, etc.
c. Detailed topography of the diversion structure site, including landforms, contours, etc.
d. Longitudinal profile and cross-sections of the river at, upstream of and downstream of the diversion structure site.

These data shall be obtained through the topographic survey procedures discussed in Part 1A of the guidelines.

9.2 Foundation Properties

Data on the foundation properties shall be sufficient to conduct stability, stress, settlement and seepage analyses required for the design of the diversion structure. For each lithologic unit present at the site, the following index and engineering properties shall be needed:

a. Index properties such as unit weight, density and coefficient of permeability.
b. Deformation modulus.
c. Static strength properties such as allowable bearing pressure and shear strength, including the shear strength along the rock-structure interface.
d. Dynamic strength properties including elastic modulus and Poisson's ratio.

These properties shall be derived from the laboratory and field procedures discussed in Part 1B of the guidelines. The deformation modulus thus obtained shall account for the effect of rock inhomogeneity, due partially to rock discontinuities, on the foundation behavior. Care shall be exercised to ensure that the strength properties of discontinuities and the weakest foundation materials, i.e. soft zones in shears and faults, are not missed. To account for uncertainties, a lower and upper bound for the foundation modulus shall be needed for each rock type modeled in the dynamic analyses.

9.3 Hydrology, Meteorological and Sedimentology

Hydrological, meteorological and sediment data shall be needed to prepare the layouts and profiles of different components of the diversion structure. These data shall consist of the following:

a. Discharges for floods of different return periods, including the Probable Maximum Flood (PMF), and the corresponding flood levels.
b. Discharge due to Glacial Lake Outburst Flood (GLOF), if applicable, and its anticipated flood level.
c. River stage and discharge at the site.
d. Normal water level (NWL) or full supply level (FSL) and high flood level (HFL).
e. Wind velocities and direction.
f. Estimate of the quantity of suspended sediments and bed loads in the river.

The hydrological and sediment data shall be obtained through procedures discussed in Parts 1C and 1D, respectively, of the guidelines. Wind data shall be the most reliable data available that are applicable to the site. They shall be obtained from site measurements.

9.4 Seismological Parameters

Seismological parameters shall be required for stability and stress analysis of the diversion structure. The principal parameters needed for these analyses include the following:

a. Controlling maximum credible earthquake (MCE) and operating basis earthquake (OBE).
b. Design response spectra for the controlling earthquakes.
c. Appropriate acceleration-time records compatible with the design response spectra.

These data shall be established through procedures described in Part 1E of the guidelines.

9.5 Environment

Environment-related data shall be needed to assess the upstream submergence due to water impoundment, downstream hazard potential due to failure of the diversion structure and the downstream riparian needs. They shall also be required to design fish passages. The principal data for these purposes shall consist of the following:

a. Settlements, agricultural lands, cultural, religious or archaeological sites, etc. around the headworks area.
b. Downstream water uses such as irrigation, water mills, etc.
c. Types of migrating fish in the river.
d. Navigability of the river before the construction of diversion structure.

9.6 Structural Materials

Data on the properties of structural materials shall be required for stability analysis, stress analysis and structural design. For concrete diversion structures, these materials shall include concrete and reinforcing steel.

9.6.1 Concrete Properties

For each grade of concrete used in the diversion structure, the following properties shall be required:

a. Unit weight.
b. Compressive, tensile and shear strengths.
c. Elastic properties such as instantaneous modulus of elasticity, Poisson's ratio and creep.
d. Thermal properties such as coefficient of thermal expansion, thermal conductivity, specific heat and diffusivity.

For preliminary design, values of these properties may be obtained from existing sources of information on similar materials. However, for final design, these values shall be obtained from laboratory testing and field investigations. The compressive strength shall be determined from the standard unconfined compression test excluding creep effects. Likewise, the tensile strength shall be determined from the modulus of rupture test or the splitting tension test, bearing in mind that the former provides results consistent with the assumed linear elastic behavior used in design. The shear strength along construction joints or at the interface with the rock foundation may be estimated by the linear relationship

Eq. 1
$$\tau = c + \sigma \tan \phi$$

where τ is the shear stress in MPa, c is the cohesion in MPa, σ is the normal stress in MPa and ϕ is the angle of internal friction.

The instantaneous modulus of elasticity of concrete shall be determined from stress-strain curves obtained from unconfined compression tests, while its Poisson's ratio shall be found from measurement of axial and lateral strains during these tests. Approximate values for creep shall generally be based on reduced values of the instantaneous modulus. However, when design requires more exact values, creep shall be based on the standard test for creep of concrete in compression.

Concrete properties required for linear elastic dynamic analysis, namely unit weight, Young's modulus of elasticity and Poisson's ratio, shall be based on concrete having sufficient age to represent the ultimate concrete properties as nearly as practicable. One-year-old specimens shall be preferred. Usually, upper and lower bound values of Young's modulus of elasticity shall be found to bracket the possibilities.

Compressive and tensile strengths required to evaluate the results of the dynamic analysis shall be obtained from the methods mentioned above. The static tensile strength determined by the splitting tensile test may be increased by 33 percent to be comparable to the standard modulus of rupture test.

To study crack initiation within mass concrete using linear finite element analysis, the value of tensile strength shall be derived from the modulus of rupture test. This strength shall be increased by 50 percent when used with seismic loading to account for rapid loading.

9.6.2 Reinforcing Steel

The yield strength of each grade of reinforcing steel shall be needed for the structural design of the concrete diversion structures. This value shall be attained from tensile strength tests of sample reinforcing bars.

9.6.3 Backfill

Backfill properties, including those of embankment fills, shall be required for computation of earth pressures against the diversion structure in stability and stress analyses. Generally, the following properties shall be needed:

a. Dry, saturated and submerged unit weight of backfills or embankment fills.
b. Angle of internal friction.
c. Cohesion.

These properties shall to be determined based on laboratory tests and field investigations discussed in Part 1B of the guidelines.

10. NON-OVERFLOW SECTION

The design of the non-overflow section shall consist of determining its length and profile, including its top elevation and upstream and downstream slopes.

10.1 Length of Non-overflow Section

The length of the non-overflow section, L_{no}, shall be determined by subtracting the required length of the overflow section, L_o, from the total waterway of the river, B, i.e.

Eq. 2 $$L_{no} = B - L_o$$

For this purpose, L_o shall be computed in accordance with Section 11.2.2. In deep and confined rivers with stable banks, B shall approximately be taken equal to the actual width of the river at the design flood. However, for meandering alluvial rivers, B shall be estimated by Lacey's equation (Varshney et al., 1982)

Eq. 3 $$B = \beta\sqrt{Q_f}$$

where Q_f is the design flood discharge in m^3/s and β is Lacey's coefficient, depending upon the hydro-geomorphological characteristics of the river channel and ranging between 4.7 to 4.9 for colluvial channels, between 4.1 to 4.7 for channels with boulder-gravel mixed soils and between 3.6 to 4.1 for rocky channels

10.2 Profile

The profile of the non-overflow section shall be fixed by finding an optimum cross-section that meets the stability and stress criteria for each loading conditions that the section is likely to be subject to. The design cross-section shall generally be established at the maximum height section and used along the rest of the non-overflow section to provide a smooth profile.

For determining the design section, a preliminary profile of the non-overflow section shall be assumed based on guidance provided in Section 10.2.1, and this profile shall be checked for its stability and state of stress through stability and stress analysis techniques discussed in Sections 14 and 15. If found unstable or overstressed, this section shall be modified and reanalyzed repeatedly till an acceptable cross-section that meets the stability and stress criteria is arrived at.

10.2.1 Selection of Preliminary Profile

The upstream face of the non-overflow section shall generally be vertical, but it may include a batter to increase sliding stability (Figure 1). Its downstream face shall usually have a uniform slope transitioning into a vertical face near the crest. In order to meet stability requirements, the downstream slope shall usually be in the range of 0.7H – 0.8H to 1V, with flatter slopes being selected for higher uplift and seismic forces.

The crest of the non-overflow section shall have sufficient thickness to resist the impact of floating objects and ice loads, if any. It shall also fulfill requirements for roadway and access to gate-operating mechanism. In any case, the top width of the crest shall not be less than 1.5 m.

The crest elevation shall accommodate the most critical combination of water surface and freeboard components determined in accordance with guidelines provided in Section 10.2.2. Accordingly, the crest shall be established at the higher of the elevations determined from the following combinations of water surface and freeboard:

a. Normal water level with normal freeboard.
b. Maximum water level with minimum freeboard.

To reduce stress concentration from seismic forces, the crest mass of non-overflow sections shall be kept to a minimum. For this purpose, curved transitions shall also be provided at slope changes.

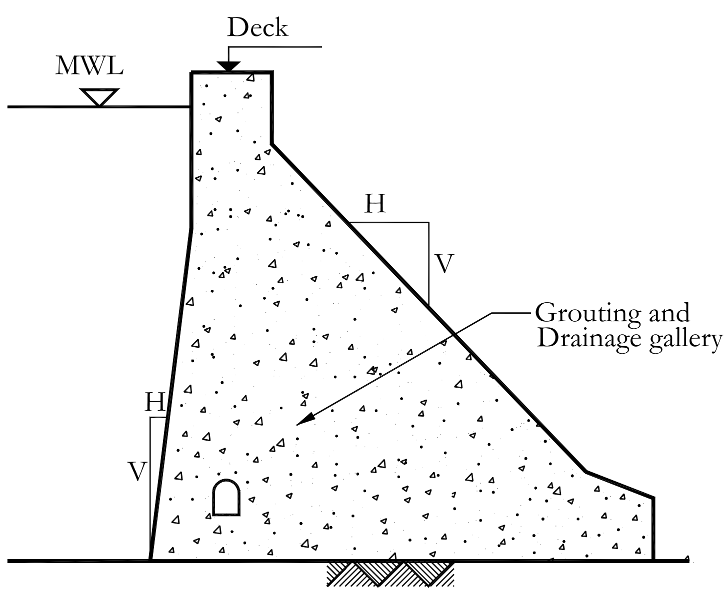


Figure 1: Typical non-overflow section of concrete diversion structure

10.2.2 Freeboard

Freeboard shall provide an adequate margin of safety against overtopping failure of the non-overflow section. It may not be necessary to prevent splashing or occasional overtopping of the concrete section by waves under extreme conditions; however, the number and duration of such occurrences shall not threaten the structural integrity of the structure, interfere with project operation or create hazards to personnel. If studies show that the non-overflow section can withstand the PMF while overtopped without significant erosion of foundation or abutment material, no freeboard may be required for the PMF condition.

10.2.2.1 Freeboard Components

The total freeboard shall include only those components of freeboard that have a reasonable probability of simultaneous occurrence for a particular water level. Generally, the following freeboard components shall be considered:

a. Wind-generated wave height and setup.
b. Waves due to earthquakes.
c. Rise of water level caused by malfunction of gates.
d. Landslide-generated waves and/or displacement of pondage volume.

10.2.2.2 Computation of Freeboard

The minimum freeboard, H_{fm}, shall be determined from the relation (Japanese National Committee on Dams, 1976)

Eq. 4
$$H_{fm} \geq \max\left(h_w + w_s, 0.5h_e\right) + h_a + h_l$$

where h_w is the wave height due to wind, w_s is the wind setup, h_e is the height of wave due to earthquakes, h_a is the rise of water level due to malfunctioning of overflow gates and h_l is the height of wave due to landslide.

Similarly, the normal freeboard, H_{fn}, shall be computed from the relation

Eq. 5
$$H_{fn} \geq h_w + w_s + 0.5h_e + h_a + h_l$$

10.2.2.3 Wave Height due to Wind

The wave height due to wind, h_w, and the wind setup, w_s, shall be determined using values of effective fetch, wind velocity over water and standard wave height. Saville's method may be used for this purpose.

10.2.2.4 Wave Height due to Earthquake

The wave height due to earthquakes, h_e, shall be evaluated from the equation (JNCD, 1976)

Eq. 6
$$h_e = \frac{\alpha_h \tau}{2\pi}\sqrt{gH_0}$$

where α_h is the coefficient of horizontal acceleration for the maximum credible earthquake, τ is the period of seismic waves in second, g is the acceleration due to gravity in m/s^2 and H_0 is the depth of reservoir water in m.

10.2.2.5 Wave due to Gate Malfunction

The rise in water level h_a due to malfunctioning of overflow section gates may be considered to be in proportion to the flood discharge and the duration of malfunction and in inverse proportion to the surface area of the pondage and the number of gates. Due to uncertainties in its prediction, h_a shall be estimated by judgment; however, its standard value may be taken as about 0.5 m.

10.2.2.6 Wave due to Landslide

Freeboard allowance for wave and volume displacement due to potential landslides which cannot be economically removed or stabilized shall be considered if a reservoir is located in a topographic setting where the wave or higher water resulting from displacement may be destructive to the diversion structure or may cause serious downstream damage. For minimum freeboard, this allowance shall be made only for cases where landslides could be triggered by the occurrence of higher water elevations and intense precipitation associated with the occurrence of the inflow design flood.

10.2.2.7 Minimum Freeboard

In any case, the height of the freeboard for non-overflow sections shall not be less than the values given in Table 1.

Table 1: Minimum freeboard for non-overflow sections

Height of structure (m)	Freeboard (m)
Less than 50	1.0
50 to 100	2.0
Over 100	2.5

(Source: Japanese National Committee on Dams, 1976)

11. OVERFLOW SECTION

The overflow section of the concrete diversion structure may be an ogee-shaped or a broad-crested weir. The design of this section shall consist of determination of its overall profile and hydraulic and structural design of its components.

11.1 Profile

The upstream face of the overflow section shall have the same configuration as the non-overflow section. To prevent damage to the concrete structure and stoplog grooves from the impact of gravel, stones and boulders, a vertical upstream face of the overflow section shall be avoided. In general, the upstream face of the section shall have a slope of 1V:1H or milder down to a level of three to five metres below the overflow crest.

The downstream face of the overflow section shall be an inverted S-shaped (Figure 2). Its upper curve shall conform closely to the profile of the lower nappe of a ventilated sheet falling from a sharp-crested weir. Flow over the crest shall be made to adhere to the face of the profile by preventing access of air to the underside of the water sheet. The profile below the upper curve of the ogee shall be continued tangent along a slope to support the water sheet on the face of the overflow. A reverse curve at the bottom of the slope shall turn the flow onto the apron of a stilling basin.

The profile of the overflow section shall be designed in a manner similar to that of the non-overflow section. It shall comply with the stability and stress criteria discussed in Section 10.2.

11.2 Hydraulic Design

The hydraulic design of the overflow section shall ensure that the section controls flows and conveys them from the pool to the tail water for all flood discharges up to the inflow design flood (IDF). The design shall consist of determining the length of the overflow section for flood flows and ice passage, its crest elevation and profile, discharge channel and terminal structure.

11.2.1 Selection of Design Parameters

The principal parameters required for the hydraulic design of overflow sections include the IDF, the permissible specific discharge over the section and the limiting flow velocity over the section.

11.2.1.1 Inflow Design Flood

The IDF shall be selected based on the consequences of failure of the diversion structure. The primary consequences that shall be considered are:

- Extent of the potential damage to life and property downstream.
- Loss of project facility.
- Loss of project revenue.

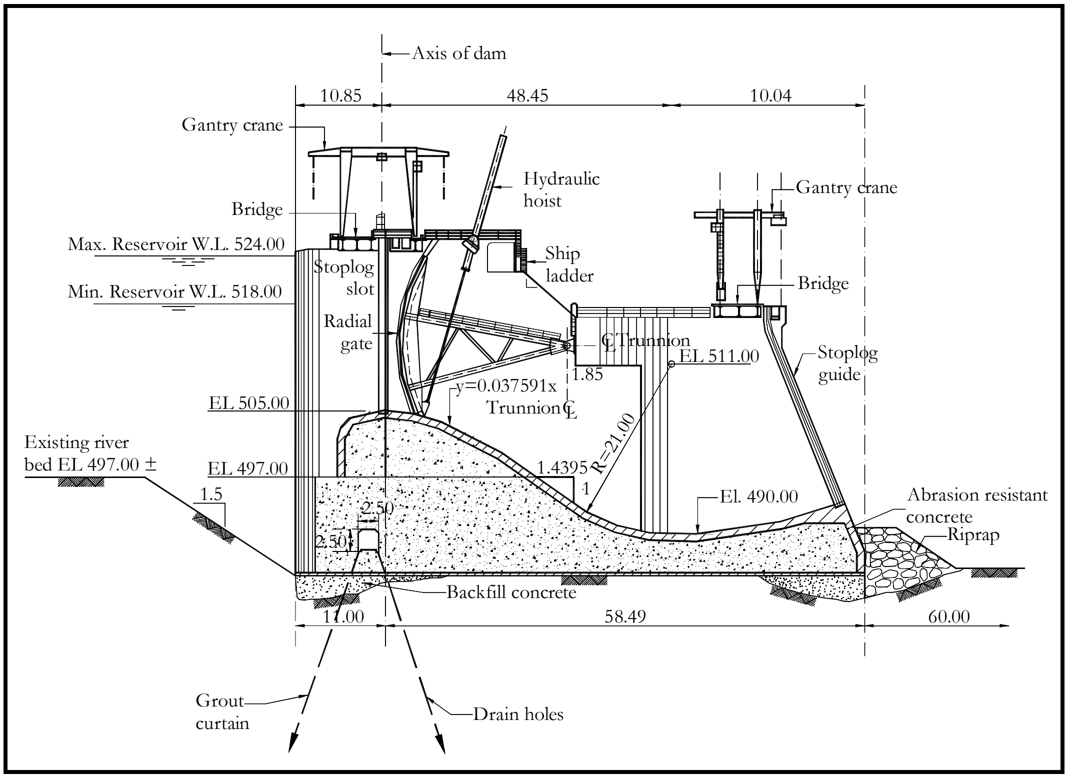


Figure 2: Overflow section of Kali Gandaki "A" Hydroelectric Project (NEA, 2002)

The relative importance of these factors shall determine the degree of conservatism to be adopted in the selection of the IDF. Conservative design criteria shall be adopted if the failure of the diversion structure cannot be tolerated because of the possible loss of life and the potential damage to property that could approach disastrous proportions. However, a much less conservative design basis may be resorted to if failure of the diversion structure neither jeopardizes human life nor creates damages beyond the financial capability of the project developer. Accordingly, the guidelines in Table 2, which are based on the potential hazards due to the failure of the diversion structure, may be used for selecting the IDF.

Table 2: Recommended inflow design floods

Hazard potential	Inflow design flood
Failure of the diversion structure would increase danger to human life and result in heavy property damage	Probable maximum flood (PMF) or Glacier Lake Outburst Flood (GLOF)
Failure of the diversion structure would not increase danger to human life but would endanger the continued operation of responsible organization or would cause heavy property damage	10,000 year frequency flood
Failure of the diversion structure would not jeopardize human life, the continued operation of responsible organization or heavy property damage	1,000 year frequency flood
Minor structure where it is permissible to anticipate failure within the useful life of the project	100 year frequency flood

The magnitudes of the PMF, GLOF or floods of different return period shall be selected based on procedures described in Part 1B of the guidelines.

11.2.1.2 Permissible Specific Discharge

The magnitude of permissible specific discharge over the crest of the overflow section shall be fixed considering the topographical, geological condition of the headworks site, type of energy dissipation, downstream flow transition regimes and on the basis of technical and economic calculations. Generally, the specific discharges for spillways on different foundation conditions shall be selected from the ranges listed in Table 3.

Table 3: Permissible specific discharges over spillway crests

Type of foundation	Permissible specific discharge (m^2/s)
Non-cohesive and cohesive soils	30 – 50
Semi-rocky foundation (mergel, argelite, etc)	50 – 90
Rocky foundation (gneiss, schist, quartzite etc.)	90 – 120

(Source: VNIIG, 1976)

11.2.1.3 Limiting Velocity

To avoid cavitation and erosion damage to the glacis of the overflow section, flow velocities on the overflow section in excess of 15 m/s shall be avoided. Where this is not possible, preventative measures to check the occurrence of possible cavitation and erosion damage shall be adopted.

11.2.2 Length and Crest Elevation of Overflow Section

For the known NWL and HFL, the length of the overflow section shall be computed as

Eq. 7
$$L_o = \frac{Q_d}{q}$$

with

Eq. 8
$$Q_d = Q_{idf} - Q_p - Q_o$$

where L_o is the total length of the overflow section, Q_d is the design flood for the overflow section, q is the permissible specific discharge, Q_{idf} is the inflow design flood, Q_p is the discharge diverted for power generation and Q_o is the discharge diverted for other purposes.

With L_o determined, the final length of the overflow section, the number of opening in it and the type and shape of the piers between openings shall be fixed considering specific requirements of the headworks.

For the known NWL, Q_d, number and types of the openings in the overflow section, shape of the piers and abutments, the crest elevation of the overflow section shall be fixed by subtracting the head over the crest, calculated using Eq. 14 , from the NWL.

11.2.3 Forms of Abutments, Wing Walls and Other Structures

Forms of abutments, wing walls and other structures connecting the overflow section with the river banks or the non-overflow section shall be carefully chosen to avoid contraction of flow and vortex zones at the overflow bays. This shall be particularly important for low and medium head diversion structures with high specific discharge and for diversion structures in which the non-overflow section is an earthfill embankment.

For preliminary selection of upstream abutments, wing walls of elliptic form shall generally be considered. The height of the wall may be kept constant throughout it length or may be decreased towards the upstream side. The length of its major semi-axis for $1.75 < p/H < 4.2$ may be determined from the relationship (VNIIG, 1976)

Eq. 9 $$\frac{l}{H} = \frac{6}{\frac{p}{H} - 1}$$

where l is the length of the major semi-axis of the ellipse, H is the head over the crest of the overflow section and p is the height of the spillway dam from the river bed.

For the computed l, the length of the minor semi-axis of the ellipse may be obtained using the following relations (VNIIG, 1976):

Eq. 10 $$\frac{l}{a} = \begin{cases} 2.5 & \text{for } \eta \le 0.25 \\ 2.8 & \text{for } \eta \approx 0.5 \\ 3.0 & \text{for } \eta >> 0.5 \end{cases}$$

with

Eq. 11 $$\eta = \frac{b_i}{B_i}$$

where a is the length of the minor semi-axis of the ellipse, b_i is the length of the right or left part of the overflow section adjoining the wing wall under consideration and B_i is the respective length of the right or left part of the upstream river channel.

The elliptic form of the wing wall may then be designed using the relationship (VNIIG, 1976)

Eq. 12 $$\frac{y}{a} = 1 - \sqrt{1 - \left(\frac{x}{l}\right)^2}$$

where x and y are coordinates of a point on the ellipse.

11.2.4 Crest of Overflow Section

The crest of the overflow section may be uncontrolled or gate controlled. The profiles and discharging capacities of two types of crests shall be determined based on the guidelines in the following sections.

11.2.4.1 Uncontrolled Ogee Crests

The portion of the crest upstream from the crest apex shall be defined as a single curve and a tangent or as a compound circular curve (). Likewise, its downstream portion shall be defined by the equation (USBR, 1978)

Eq. 13 $$\frac{y}{H_o} = -K\left(\frac{x}{H_o}\right)^n$$

where H_o is the design head on the crest during design flood, K is a constant whose value depends on the velocity of approach, obtained from , and n is a constant whose value depends on the upstream inclination and on the velocity of approach, obtained from .

Discharge Capacity

The ogee shall be proportioned to discharge the inflow design flood. The discharge capacity of the ogee crest shall be determined using the formula (USBR, 1978)

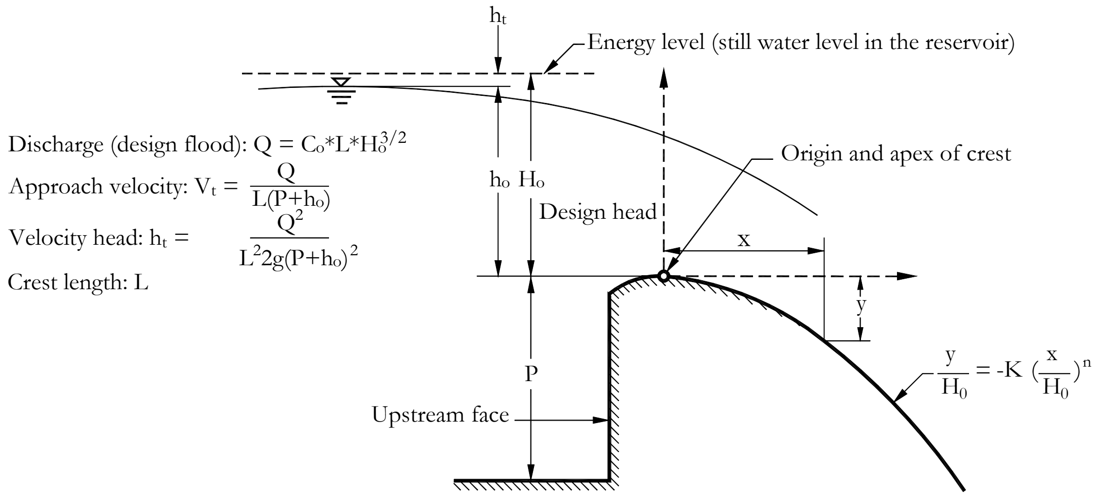


Figure 3: Elements of nappe-shape crest profile (USBR, 1978)

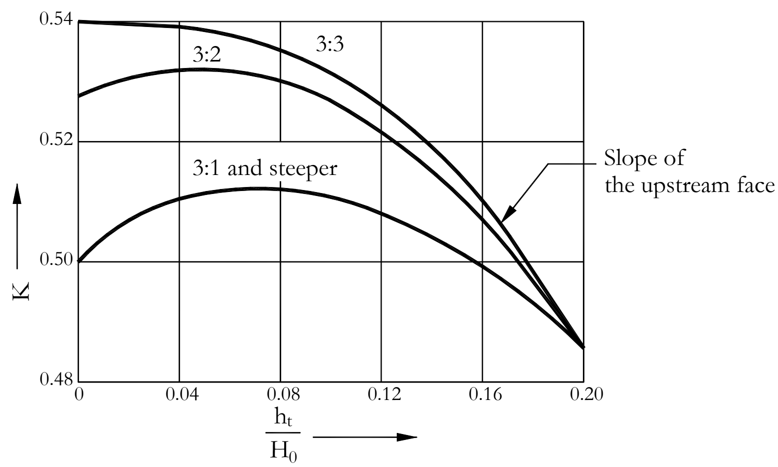


Figure 4: Values of *K* for definition of crest profile (USBR, 1978)

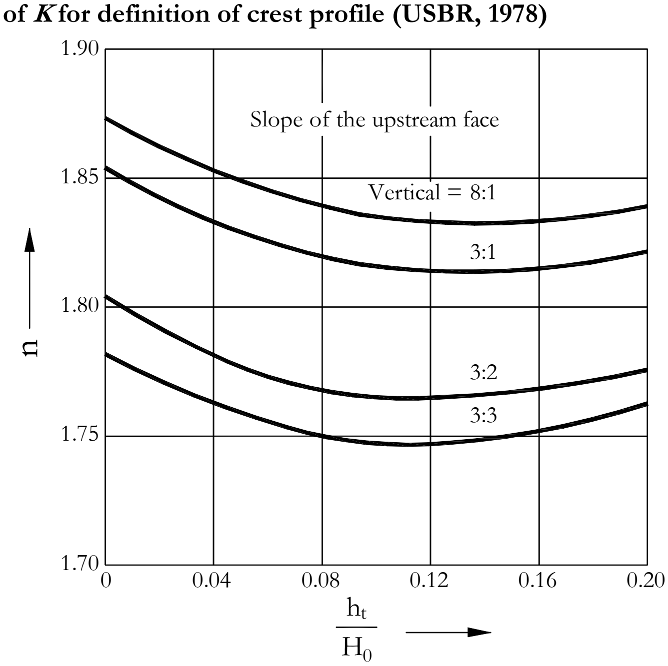


Figure 5: Values of *n* for definition of crest profile (USBR, 1978)

Eq. 14 $$Q = CLH_e^{3/2}$$

where Q is the discharge in m^3/s, L is the effective length of the crest in m obtained from Eq. 16, H_e is the total head on the crest including velocity of approach head in m and C is a variable coefficient given by

Eq. 15 $$C = k_1 k_2 k_3 k_4 C_o$$

where C_o is the discharge coefficient for head equal to design head (Figure 6), k_1 is the correction for head other than the design head (Figure 7), k_2 is the correction for sloping upstream apron (Figure 8), k_3 is the correction for tailwater effect (Figure 9) and k_4 is the correction for head other than the design head (Figure 10).

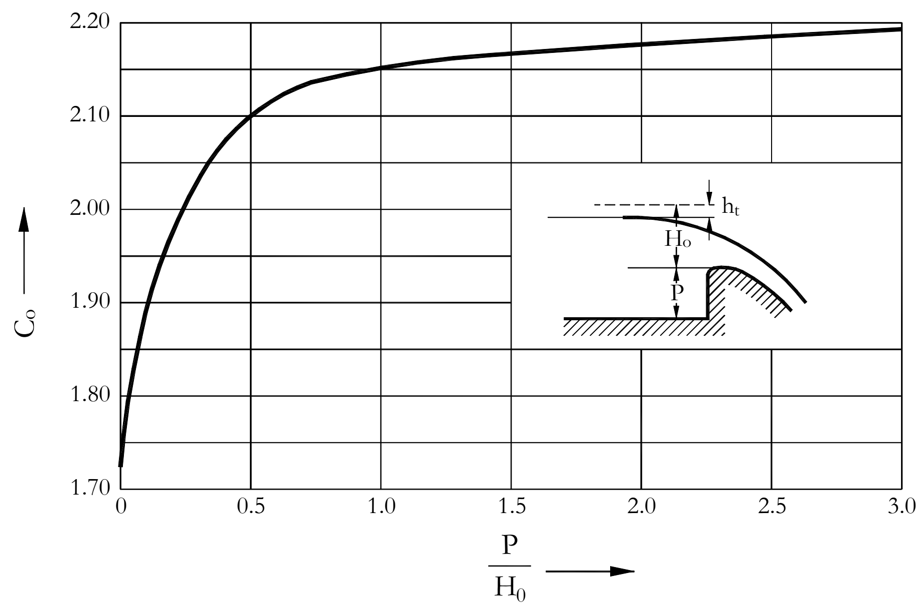


Figure 6: Discharge coefficients for heads equal to design head (USBR, 1978)

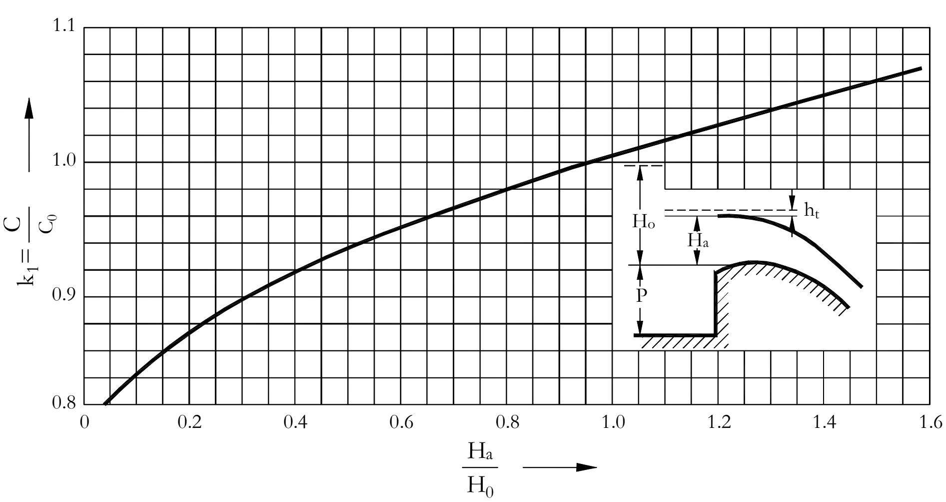


Figure 7: Correction for heads other than design head (USBR, 1978)

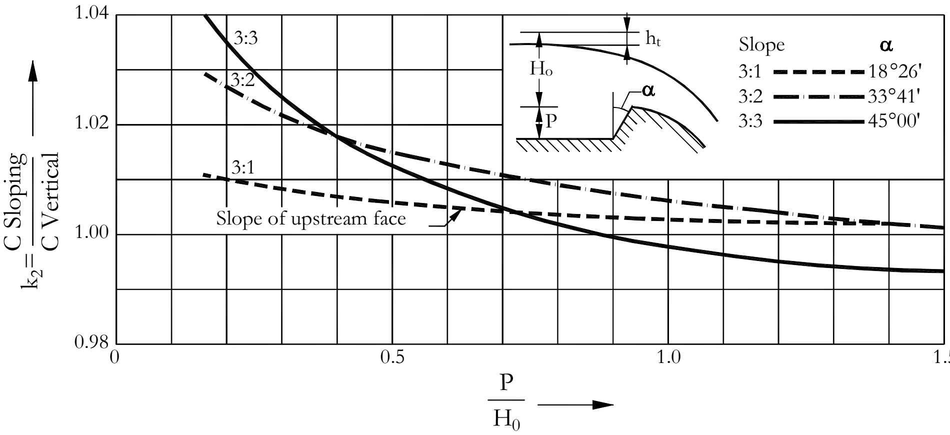


Figure 8: Coefficient of discharge of sloping upstream face (USBR, 1978)

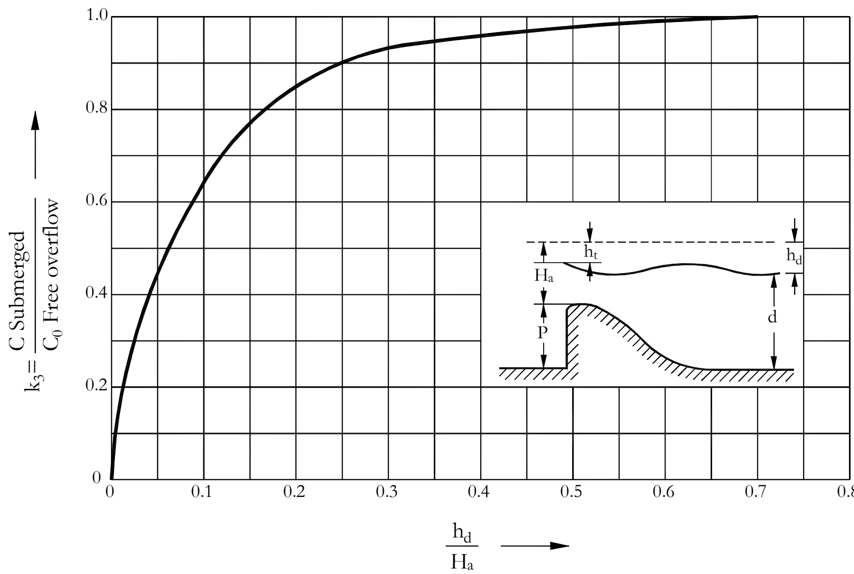


Figure 9: Correction for tailwater effect (USBR, 1978)

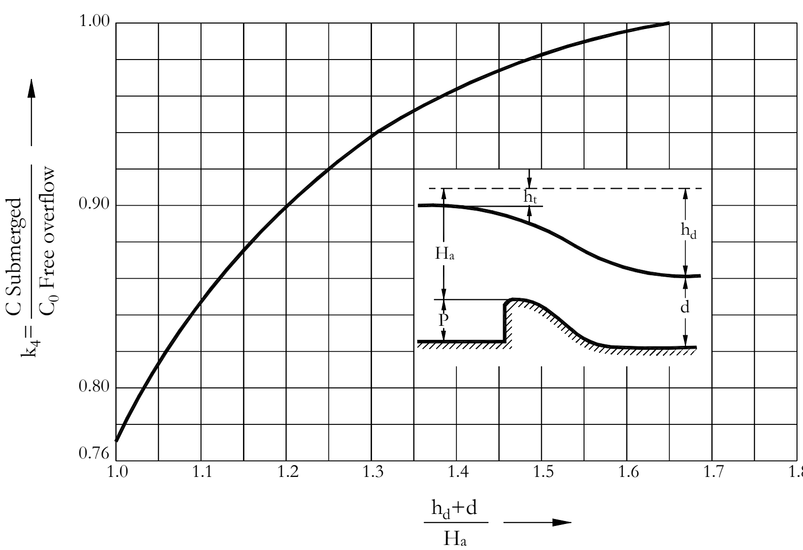


Figure 10: Correction due to downstream apron effect (USBR, 1978)

Effective Length of Crest

The value of effective length, L, used in Eq. 14 shall take into account the side contractions of the overflow due to crest piers and abutments. Accordingly, L shall be calculated using the relation (USBR, 1978)

Eq. 16
$$L = L_o - 2(NK_p + K_a)H_e$$

where L_o is the net length of the weir crest in m obtained from Eq. 7, N is the number of piers, K_p is the pier contraction coefficient, K_a is the abutment contraction coefficient and H_e is the total head on the crest, including velocity of approach, head in m.

Coefficient K_p shall be obtained from Table 4. Likewise, K_a shall be taken from Table 5 in which r is the radius of the abutment.

Table 4: Pier contraction coefficient

Pier shape	K_p
Square-nosed pier with corners rounded on a radius equal to about 0.1 of the pier thickness	0.02
Round-nosed pier	0.01
Pointed-nose pier	0

(Source: USBR, 1978)

Table 5: Abutment contraction coefficient

Abutment shape	K_a
Square abutments with headwall at 90º to the direction of flow	0.02
Rounded abutments with headwall at 90º to the direction of flow, when $0.5H_o \geq r \geq 0.15H_o$	0.01
Rounded abutments where $r > 0.5H_o$ and headwall is placed not more than 45º to the direction of flow	0

(Source: USBR, 1978)

Model Test

Hydraulic model tests on ungated ogee crest shall be performed if the direction of river flow is not at perpendicular to the crest. The tests may also be desirable if other site conditions make reliable calculation for design of the overflow section difficult.

11.2.4.2 Gate-controlled Ogee Crest

A gate-controlled ogee crest may be designed to follow the ideal nappe profile for maximum head. Alternately, its profile may follow the trajectory of a jet issuing from an orifice flow caused by releases for partial gate openings. In the latter case, the profile shall be defined by the equation (USBR, 1978)

Eq. 17
$$-y = x\tan\theta + \frac{x^2}{4H\cos^2\theta}$$

where H is the head on the center of the opening in m and θ is the angle at which the orifice is inclined from the vertical.

If the profile of the weir follows the nappe profile, sub-atmospheric pressures that may arise along the crest contact may be minimized by placing the gate sill downstream from the crest of the ogee so that steeper trajectory more nearly conforming to the nappe-shaped profile is attained.

If sub-atmospheric pressures are to be avoided along the crest contact, the shape of the ogee downstream from the gate sill shall conform to the trajectory profile corresponding to orifice flow. This shall result in a wider ogee and reduced discharge efficiency for full gate opening.

Discharge Capacity

The discharge for a gated ogee crest at partial gate openings shall be similar to flow through low-head orifice and may be computed through the equation (USBR, 1978)

Eq. 18
$$Q = \frac{2}{3}\sqrt{2g}CL\left(H_1^{3/2} - H_2^{3/2}\right)$$

where L is the effective length of the weir crest in m, g is the acceleration due to gravity in m/s^2, H_1 is the total head (including velocity head of approach) to the bottom of the orifice in m, H_2 is the total head (including velocity head of approach) to the top of the orifice in m and C is the discharge coefficient obtained from Figure 11.

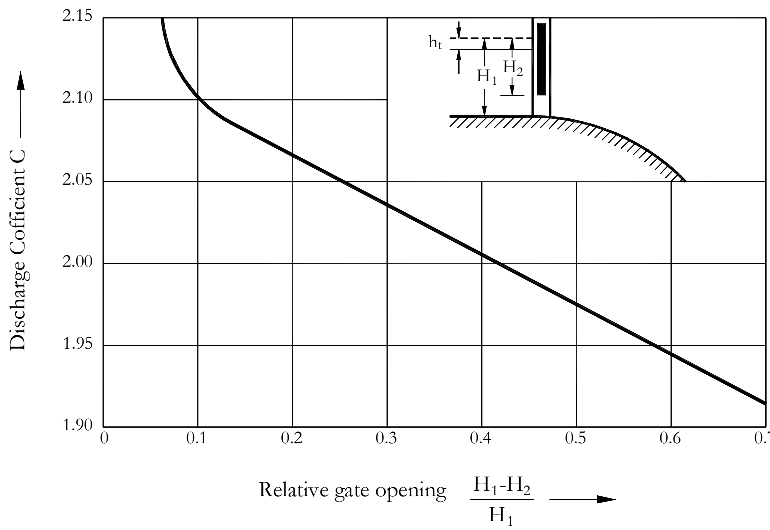


Figure 11: Discharge coefficient for flow under gate (USBR, 1978)

Eq. 18 shall be used for computing the discharge capacity of a gated crest till the gate opening is less than two-third of the upstream water depth. As a transition to free surface flow occurs for larger openings, Eq. 14 shall be used for computation of discharge capacity.

Model Test

Model tests shall be performed for an accurate determination of the discharge capacity of a gated ogee crest. These tests shall study the effects of the shape and length of the side walls, the radius and inclination of the gates and the location of the gate in relation to the crest.

11.2.5 Glacis and Flow Transition

The slope and length of the glacis shall be determined by the form and dimensions of the structure and by the scheme for transition of flow from the glacis to the downstream energy dissipation. For the surface flow transition, the curvilinear transition part shall be terminated with the nose-tramplline, the level of which shall be fixed by design or by model tests.

For a given downstream flow regime, the glacis and the downstream strengthening shall be conjugated smoothly with a cylindrical surface having radius given by (VNIIG, 1976a)

Eq. 19 $$R = (0.25 - 1.0)\left(Z + H_o\right)$$

where Z is the difference in upstream and downstream water levels and H_o is the dynamic head over the spillway crest

In order to transit overflow regime at the downstream, the height of the nose-tramplline shall equal or slightly exceed the downstream depth of flow. It shall be determined by the form of the structure and the requirements for downstream flow conditions. In this case, the radius of the tramplline cylindrical surface shall not be less than six times the flow depth at the nose-tramplline.

11.2.5.1 Flow Parameters for Glacis

The depths and velocities of the gradually varied flow over glacis shall be determined either by computer programs or by drawing the water surface profile considering energy losses and flow aeration. Aeration shall be taken into account from a section at which the mean flow velocity exceeds the velocity given by (VNIIG, 1976a)

Eq. 20 $$v_{ba} = 6.63\sqrt{gR\cos\alpha_d}\sqrt{1+\frac{0.011}{R^2}}\left(1+\frac{8.7C}{R^{1/6}}\right)^{-1}$$

where v_{ba} is the flow velocity at the section where aeration begins, R is the hydraulic mean radius, α_d is the angle formed by the glacis with the horizontal and C is Chezy's coefficient of roughness.

In absence of aeration, the water surface profile between two sections i and i+1 shall be drawn using the following set of equations (VNIIG, 1976a, b):

Eq. 21 $$h_{i+1}\cos\alpha_d + \frac{v_{i+1}^2}{2g} - h_i\cos\alpha_d - \frac{v_i^2}{2g} = \left(S - i_f\right)l$$

Eq. 22 $$h_i v_i = h_{i+1} v_{i+1}$$

where h_i is the depth of flow at section i,, v_i is the velocity of flow at section i, g is the acceleration due to gravity, $S$is the slope of the spillway glacis, l is the distance between sections i and i+1, and i_f is the mean value of the friction angle of flow in the length considered, given by (VNIIG, 1976a, b).

Eq. 23 $$i_f = \frac{v_m^2}{C_m^2 R_m}$$

where v_m is the mean velocity of flow between sections i and i+1, R_m is the mean hydraulic radius of the flow section between sections i and i+1 and C_m is the mean Chezy's coefficient, computed using the relation (VNIIG, 1976a)

Eq. 24 $$C_m = \frac{1}{C} R_m^{1/6}$$

In the initial part of the glacis up to 5 to 10 times H, where head losses can be neglected, the depth and velocity of flow shall be determined from the following equations (VNIIG, 1976a, b)

Eq. 25 $$h\cos\alpha_d + \frac{v^2}{2g} = H_o + Z$$

Eq. 26 $$q = vh$$

where Z is the difference in elevation between the crest of the overflow section and the section of the glacis under consideration.

The piezoelectric pressure, p, on the glacis shall be determined from the formula (VNIIG, 1976a, b)

Eq. 27 $$p = \gamma_w h_i \cos\alpha_d$$

where γ_w is the unit weight of water.

Calculation of the contraction and expansion of the overflow section and bends shall be conducted by relevant formulae for the turbulent flow regime.

11.2.5.2 Parameters for Flow Transition

The air saturated flow in the range of this part due to rise in pressure and deaeration is relatively no so high and cannot be considered in the design. The depth of flow and its velocity for high dams in the first approximation shall be determined with the use of the Bernoulli's equation (VNIIG, 1976b)

Eq. 28 $$\nabla Z + h_1\cos\alpha_{d1} + \frac{v_1^2}{2g} = h_2\cos\alpha_{d2} + \frac{v_2^2}{2g} + \frac{p_c}{\gamma_w} + \frac{v_m^2 l_{1-2}}{C_m^2 \mathrm{R}_m}$$

where ∇Z is the rise in level of the end section bed of the spillway glacis over the section of curvilinear part considered, h_1 is the depth of flow determined without consideration of aeration, $l_{1\text{-}2}$ is the length of the part between the considered sections, h_m is the mean value of the flow depth, v_m is the mean value of the flow velocity, R_m is the mean value of the hydraulic radius, C_m is the mean value of Chezy's coefficient and α_d is the angle formed by the glacis with the horizontal.

In Eq. 28, p_c/γ_w represents the specific centrifugal pressure on the flow surface to be determined as $R/h_1 \geq 6$ to 8 by the formula (VNIIG, 1976b)

Eq. 29 $$\frac{p_c}{\gamma_w} = \frac{h_1 v_1^2}{g\mathrm{R}}$$

When using Eq. 28 for the end section of transition, when the supply of air about the tramplline is possible,

Eq. 30 $$h_2\cos\alpha_{d2} + \frac{p_c}{\gamma_w} = 0$$

i.e., the pressure at the given section becomes equal to the atmospheric pressure.

Flow parameters for tramplines with cylindrical surface shall be determined by hydraulics of turbulent flow regime. For low and medium head diversion structures, the flow depth and velocity at the end of the transition shall usually be determined by the formulae.

Eq. 31 $$h_1 = \frac{C_d\sqrt{2g}H^{3/2}}{v_1}$$

and

Eq. 32 $$v_1 = \varphi\sqrt{2g(T_o - h_1)}$$

where T_o is the specific energy of flow behind the structure with respect to the surface of the stilling basin and/or tailwater surface level and φ is a velocity coefficient varying from 0.98 to 0.95.

11.2.6 Cavitation

The phenomena of cavitation and cavitation erosion shall be considered in the design of overflow sections which experience high flow velocities, generally in excess of 15 m/s. For this purpose, the possibility of cavitation on the crest and glacis of the overflow section due to possible sources shall be determined. Where such possibility is found to exist, suitable measures shall be adopted to eliminate the cavitation.

11.2.6.1 Sources of Cavitation

For predicting the potential for cavitation, the following sources shall be considered:

a. Geometric forms of the elements of the overflow section such as bell mouths, divide walls, gate chambers, gates and their grooves, bends, expansions, reverse humps, etc.
b. Local surface irregularities such as humps, reverse humps due to improper formworks or concrete placement, reinforcement protrusions in zones of flow, non-uniform joints, welding joints of metal linings, etc.

Prediction of cavitation due to the first source shall be made based on the geometrical characteristics of element under consideration. However, owing to their uncertainty at the design stage, the dimensions and forms of the second source shall be based on the prototype field data.

11.2.6.2 Conditions for Cavitation

Cavitation shall occur when the absolute pressure in flow falls below the pressure of vapor formation, i.e. (VNIIG, 1976b)

Eq. 33 $$p_i = p_a + p_s = p_a + \bar{p}_s - p'_s \leq p_v$$

where p_i is the absolute pressure at the point of flow considered, p_a is the atmospheric pressure at the point of flow considered, p_s is the actual hydrodynamic pressure weighted in time, $\bar{p}_s$ is the mean hydrodynamic pressure weighted in time, p'_s is the pulsated hydrodynamic pressure weighted in time and p_v is the pressure of the vapour formation of flow for the given temperature.

The values of p_a and p_v may be adopted from Table 6 and Table 7 respectively.

Table 6: Values of atmospheric pressure for different altitudes

Altitude amsl (m)	0	200	500	1000	1500	2000
Atmospheric pressure, p_a (MPa)	1.05	1.02	0.99	0.94	0.88	0.83

(Source: (VNIIG, 1976b)

Table 7: Values of pressure of vapour formation

Water temperature, °C	10	15	20	25	30
Pressure of vapor formation, p_v (MPa)	0.013	0.017	0.024	0.033	0.045

(Source: (VNIIG, 1976b)

Alternatively, cavitation may be assumed to occur when (VNIIG, 1976b)

Eq. 34

$$K_i \leq K_{cr}$$

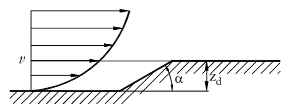
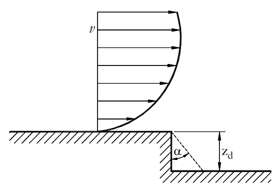
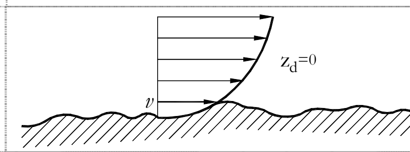
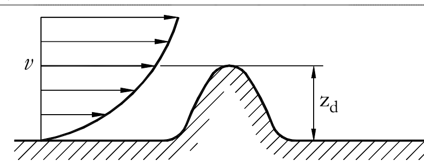
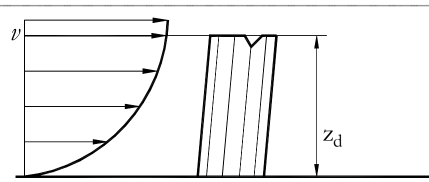
where K_{cr} is the critical cavitation parameter defined for some elements in Table 8 and K_i is a cavitation parameter characterizing hydrodynamic condition of flow in relation between the pressure and flow velocity at the considered point, defined as

Eq. 35

$$K_i = \frac{p_i - p_v}{0.5\rho v_i^2} = \frac{p_{cb} - p_v}{0.5\rho v_{cb}^2}$$

where p_{cb} is the characteristic pressure adopted as the design magnitude for given point of flow, v_{cb} is the characteristic flow velocity adopted as the design magnitude value for given point of flow and ρ is the flow density.

Table 8: Typical values of K_{cr}

Types of irregularity	**Sketch of irregularity**	K_{cr}
Hump to flow		For 90° ≤ α ≤ 5° $0.466\sqrt[3]{\alpha}$
Reverse hump to flow		For 90° ≥ α >20° 1 for $z_d \geq \delta$, $(z_d/\delta)^{3/4}$ for $z_d < \delta$, δ being the thickness of the boundary layer
Uniform natural roughness of surface with average height of humps, Δ		1.0
Single hump with sharp upper edge throughout section		2.0
Separated exposing local irregularity		3.5

(Source: (VNIIG, 1976a, b)

11.2.6.3 Methods for Elimination of Cavitation

As satisfaction of conditions in Eq. 33 and Eq. 34 in the design of different elements of the overflow section is not generally possible, the preventive measures discussed below may be considered for eliminating or damping the cavitation phenomenon in overflow structures.

Selection of Appropriate Geometric Form of Elements

Cavitation of any element of the overflow section may be achieved by selecting a suitable geometric form for it. For this purpose, various alternative forms of the element shall be considered.

Change in Working Condition of Overflow Section

Cavitation may be eliminated either by increasing the atmospheric pressure of flow or by reducing its velocity. For this purpose, the form of the overflow section shall be designed such that the hydraulic regime of operation of the overflow section corresponds to these conditions. For this purpose, the following measures may be applied:

a. Decreasing the slope of the non-pressure parts of the overflow section
b. Provision of work of the spillways with full opening of gates and limitation of partial opening of gates.

As early mentioned, cavitation can be eliminated, either by increase in the atmospheric pressure of flow or by reduce its velocity. In connection with this, the selection of hydraulic regime of work of the spillways is the most essential, that is determined by its design of form of the spillway structures to a large extent.

Air Saturation of Flow along Overflow Section

Cavitation may be effectively eliminated by raising the pressures in the potential zones of low pressure through supply of air to these zones. This may be achieved through air supply on the flow path.

The dimensions of arrangements for reverse hump to flow may be determined by the dimensions of the elements of the overflow section protected with the supply of the air. Its discharge may approximately be found from the formula (VNIIG, 1976a)

Eq. 36
$$Q_{air} = 0.1 l_y h_y v_m$$

where Q_{air} is the discharge of air to be catching in the zone of low pressure in comparison with the atmospheric pressure, l_y is the linear dimension of the element to be protected from cavitation or of the zone of low pressure (width of overflow section or its bay, height of the walls of overflow section, etc.), h_y is the dimension of the deflector exposed to flow (reverse hump height, width of semi-groove, difference in radii of diaphragms) and v_m is the mean velocity of flow of water.

In other cases, Q_{air} shall be determined experimentally.

11.2.7 Protection of Overflow Surface from Abrasion

In view of the large amounts of bed sediments passing over the overflow section, the entire surface of the overflow section shall be protected from sediment-induced abrasion. For this purpose, one of the following abrasion-resistant covers may be provided over the surface:

a. High quality and abrasion resistant concrete.
b. High performance micro-silica concrete.
c. Steel-fibre reinforced high quality concrete.
d. Steel lining.
e. Steel rails embedded in concrete.
f. Dressed hard-stone masonry lining (granite, quartzite, basalt, serite, etc.).
g. Epoxy coating.
h. Rubber lining.

11.2.8 Terminal Structures

Some of the common terminal structures suitable for concrete diversion structures are the hydraulic jump stilling basins, roller buckets and deflector buckets. Guidance on selection and design of these structures is presented in Part 2C of these guidelines.

11.3 Structural Design of Crest

Structural design of the overflow crest shall aim at determining the reinforcement required to resist the tensile stresses generated by the loads acting on the crest.

11.3.1.1 Design Load Condition

The crest shall be designed for the load cases discussed in Section 14.2. For design, tensile stress resulting from the most critical of the load conditions listed in Section 14.3 shall be considered. However, these shall be calculated for the following three regions (Figure 12):

a. Considering block as a whole for section 1-1 and below.
b. Region near the pier (at section 2-2 located above section 1-1).
c. Region away from the pier (at section 2-2 located above section 1-1).

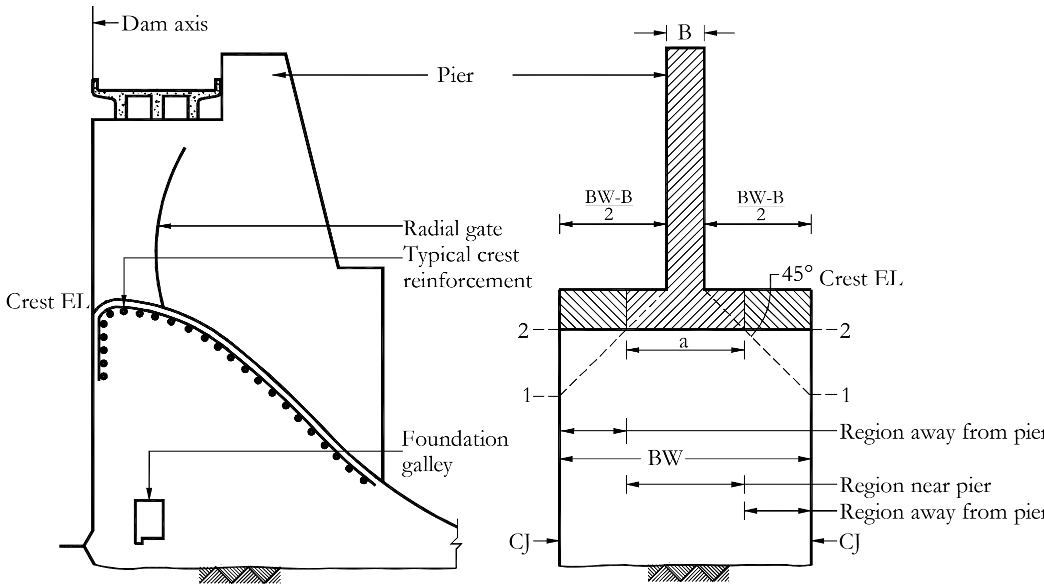


Figure 12: Sections for computation of stresses in crest

11.3.2 Computation of Forces

The self weight of the crest and pier shall be calculated for the areas above the section under consideration.

For discharging condition, the weight of water over the crest shall be ignored for tail water levels below crest. If the design head is less than the head corresponding to MWL, negative pressures over the crest shall be considered.

For gated crests, the horizontal water pressure acting above the crest elevation over the gates shall be taken to be transferred to the pier at an appropriate elevation depending on the type of gates used. This force shall not be considered for regions away from the pier in respect of sections above 1-1.

Uplift pressures shall be considered to be acting over the overflow section only and not in the pier portion. For spillway discharging conditions and tail water levels below the section under consideration, the effect of the sheet of water flowing over the crest for uplift calculations may be ignored. For tailwater levels above the section considered, uplift at the downstream end and the weight of water above crest shall be suitably considered.

11.3.3 Minimum Concrete Thickness

The minimum thickness of structural concrete at the crest shall be l.5 m measured normally. This thickness has to be suitably increased to accommodate the anchorage below the piers.

11.3.4 Reinforcement

Crest reinforcement shall be provided only in regions where the computed tensile stresses exceed the allowable tensile stresses of the concrete. The reinforcement shall be properly anchored in regions of compression. Distribution reinforcement equal to not less than 20% of the main reinforcement shall be provided. However, it shall not be less than 16 mm diameter at 250 mm centers.

12. UNDERSLUICE

The undersluice shall be provided to flush out the sediments deposited in front of the intake and thus control the bed levels in its approach area.

12.1 Hydraulic Design

Hydraulic design of the undersluice shall consist of determining its location, length, profile and opening size.

12.1.1 Location

The undersluice shall be located close to the intake. It shall be provided in continuation with the overflow section, separated from the latter by a divide wall.

12.1.2 Length of Undersluice

The length of the undersluice shall be fixed based on the following considerations:

a. It shall be capable of passing at least twice the diverted discharge to ensure efficient flushing capacity.
b. It shall be capable of passing about 10 to 20 percent of the maximum flood discharge at high floods.
c. It shall be capable of passing fair weather freshets and low monsoon floods for obviating overtopping and/or operation of crest gates.

12.1.3 Bays and Bay Openings

Based on the flow to be passed, the undersluice may consist of one or more bays. However, for small rivers flows, a single sluice bay may be preferred over two or more bays with smaller openings.

The bay opening of the undersluice shall be sized to pass the largest possible boulders brought along by the river. Generally, the minimum opening shall be twice the boulder size expected in the river stretch in the vicinity of the headworks.

12.1.4 Profile

The undersluice shall be designed as a broad-crested weir with downstream submergence. Its crest level shall be maintained close to the general ground level, about 2 m below the intake invert level. The design shall be performed in accordance with the design of a flumed section. The capacity of the undersluice shall be sufficient under submergence conditions.

12.1.5 Energy Dissipation

Normally, energy dissipation downstream of the undersluice shall be achieved through a stilling basin with a downstream cutoff to check undermining. The basin shall be designed in accordance with the provisions of Part 2C of these guidelines.

12.2 Structural Design

Structural design of the undersluice shall be performed in accordance with the provisions for the overflow section. In some cases, the undersluice may be designed as a reinforced cement concrete raft.

For protection against abrasion from rolling boulders, the surface of the undersluice shall be lined with abrasion resistant material (refer Section 11.2.7). Similarly, the vertical faces of divide walls and other structures abutting against the undersluice surface shall be provided with abrasion-resistant material up to a minimum height of 1 m above the undersluice surface.

13. CONSIDERATIONS FOR PERMEABLE FOUNDATIONS

Diversion structures constructed on pervious foundations (boulder-gravel mixed soil) shall be designed with due safety against scour of the foundation material caused by seepage under the structure.

13.1 Measures for Scour and Seepage Control

Measures adopted for control of scour, seepage and uplift forces under the structure shall depend on the type, stratification, permeability, homogeneity and other properties of the foundation materials and the size as well as the physical requirements of the structure itself. Generally, these measures shall consist of some, all or various combinations of the following devices (Figure 13):

a. Upstream apron, with or without cutoffs at its end.
b. Downstream apron, with or without cutoffs at its end, and with or without filters and drains under the apron.
c. Cutoffs at the upstream or downstream end or at both ends of the diversion structure or control section, with or without filters or drains under the section.

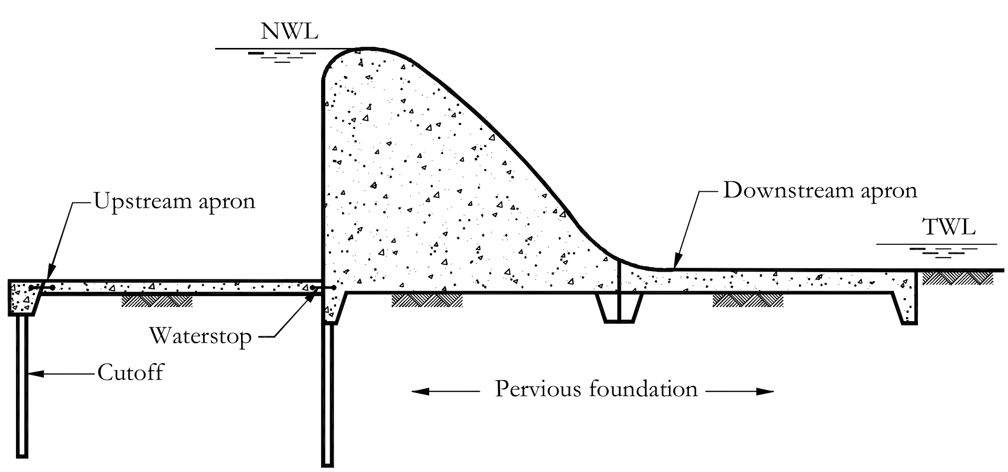


Figure 13: Aprons and cutoff for diversion structures on permeable foundations

13.1.1 Aprons

A concrete apron may be placed upstream of the diversion structure in conjunction with one of the various types of cutoff walls to increase the length of the path of seepage, thereby reducing uplift under the main portion of the diversion structure. The apron shall usually be connected to the diversion structure and to a concrete cap over the piling with flexible water stops, which allow differential movement to take place without accompanying detrimental cracking. The safety of the structure may be further improved by placing an impervious earth blanket over a portion of the concrete apron and the streambed upstream from it.

Downstream concrete aprons may be provided to lengthen the path of seepage in the foundations and also create a basin where the surplus energy of the overflowing water may be safely dissipated. In cases where it is not feasible to construct a concrete apron of sufficient length to avoid erosion entirely, additional protection may be gained by placing riprap downstream from the apron.

13.1.2 Cutoff Walls

Cutoff walls may be constructed of concrete, cement-bound curtains, steel sheet piling or impervious earth compacted in a trench. The type of cutoff shall depend upon the type of the foundation material and the depth of the cutoffs.

Concrete cutoffs may be used under aprons or under the diversion structure at sites where under-seepage is large. It may also be used under the diversion structure to increase the sliding resistance of the structure.

Sheet piling cutoffs of interlocking steel sections may be used under the aprons of diversion structures. They shall be anchored to the apron or diversion structure by means of concrete caps.

13.1.3 Filters and Drains

Relief of uplift pressures under the apron or downstream toe of the diversion structure may be accomplished by drains. Drains shall generally be of sewer pipe laid in graded material which acts as a filter. They may be perforated pipe or plain pipe laid with open joints. The drains may be located at the downstream toe of the diversion structure, at selected places under the downstream apron and immediately upstream from the downstream cutoff.

Weep holes shall normally be used for relief of uplift pressure under aprons and behind walls. To prevent piping, the gradation of the filter materials used in conjunction with the weep holes shall be carefully selected with respect to the gradation of the foundation materials. Both uniform grain-size and graded filters may be used.

13.2 Hydraulic Design

Hydraulic design of the aprons and cutoffs shall be aimed at proportioning these elements to control the amount of seepage under the diversion structure and to limit the intensity of the uplift so that the stability of the structure is not threatened.

13.2.1 Computation of Uplift Pressure

The uplift pressure at any point under the diversion structure shall be calculated by any accepted practice. The calculation shall take into account several factors such as the head on the diversion structure, depth of the permeable strata below the structure, permeability of the foundation material, length of upstream and downstream aprons, depths and tightness of cutoffs and effectiveness of the drains. In general, the uplift pressure at any point under the diversion structure may be computed using Lane's weighted creep method, the flow net method, Khosla's theory of independent variables or the finite element method.

13.2.2 Apron

The total lengths of the upstream and downstream aprons shall be fixed in conjunction with the depth of downstream cutoff to satisfy the requirements of exit gradient, scour and economy.

13.2.3 Cutoffs

The upstream and downstream cutoffs shall generally be provided to cater for scours up to 1 to 1.25 times the depth of scour, respectively. For foundation material, the depth of scour may be calculated as per the provisions for downstream protective apron in Part C of these guidelines.

13.2.4 Filters and Drains

Filters and drains shall be designed in accordance with the provisions described in Part C of these guidelines.

13.2.5 Upstream Protection Works

The river stretch upstream of concrete diversion structures built on pervious foundations shall be protected against scour. The length of upstream protection shall approximately be equal to the design depth of scour below the floor level.

Upstream scour protection may be achieved through one of the following means:

a. Placing boulders over the slope of the scour trench from the dam site.

b. Placing concrete blocks, flexibly connected in the longitudinal and transverse directions, over two to three layers of boulder soling.

13.2.6 Downstream Protection Works

Downstream protective works shall be designed in accordance with the provisions described in Part C of these guidelines.

13.3 Structural Design

Structural design of aprons and cutoffs shall essentially consist of proportioning them and to enable them to safely resist all forces acting on them. In reinforced concrete elements, computation of reinforcement shall also be performed.

13.3.1 Cut-offs

13.3.1.1 Design Load Conditions

Cutoffs shall be designed to resist the worst combination of forces and moments acting on them. The principal consideration in computing these forces and moments shall include the following:

a. Possible scour on the outer side.

b. Earth pressure and surcharge due to floor loads on the inner side.

c. Differential hydrodynamic pressure computed on the basis of the percentage of pressure of seepage flow below the floor.

13.3.1.2 Steel Sheet Piles

The steel sheet pile cutoff shall be designed as sheet pile retaining walls anchored at its top. If its effect in resisting the forward sliding of the structure is taken into account, the cutoff shall also be designed to withstand the passive pressures thus developed. The reinforced concrete pile caps shall be designed to transmit the forces and moments acting on the steel sheet pile cut-offs to the aprons or the diversion structure.

13.3.1.3 Reinforced Concrete Walls

Reinforced concrete cutoffs shall be designed to resist forces and moments as a cantilever wall cast monolithically with the apron floor or the diversion structure. The horizontal load on the cantilever shall be taken as the load that is in excess of the foundation's resistance to sliding.

13.3.2 Aprons

Upstream and downstream aprons shall be gravity type impervious floors that shall balance the uplift pressure by the self weight of the floor only. They shall generally be constructed in plain concrete with suitable temperature reinforcement.

The thickness of the aprons shall be adequate to counter balance the uplift pressure at the point under consideration under the worst possible combination of loads in different conditions, including seismic conditions. The thickness adopted for construction shall be at least 10 percent greater than the thickness required to counteract the uplift pressure

14. STABILITY ANALYSIS

Stability analysis of the non-overflow and overflow sections of concrete diversion structures shall be conducted to check their margin of safety against overturning, sliding, flotation and overstressing.

14.1 Stability Requirements

For all load conditions they are likely to be subject to during their lifetime, the non-overflow and overflow sections shall satisfy the following stability requirements:

a. The section shall be safe against overturning at any horizontal plane within it, at its base or at a plane below its base.
b. The section shall be safe against sliding on any horizontal or near-horizontal plane within it, at its foundation or within the foundation.
c. The allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

Characteristic locations within the diversion structures at which stability criteria checks shall be made include locations of cross-sectional changes and high concentrated loads. Large galleries and openings within the structures and the locations of upstream and downstream slope transitions shall be specifically considered in the stability analysis.

14.2 Design Loads

Loads chosen for stability analysis of concrete diversion structures shall represent the actual loads that shall act on the structure. Generally, the following loads shall be considered:

a. Dead load.
b. Headwater and tailwater pressures.
c. Uplift pressure.
d. Nappe pressure.
e. Earthquake forces.
f. Earth pressure.
g. Silt pressure.
h. Wind pressure.
i. Wave pressure.
j. Thermal loads.
k. Reaction of foundations.

As far as possible, loads shall be exactly determined from known properties of materials and fluids. Loads which are not amenable to exact determination shall be estimated based on consideration of all available facts and, to a certain extent, on judgment and experience.

14.2.1 Dead Loads

Dead load shall include the weight of concrete, superimposed backfill and appurtenances such as gates and bridges. While determining these loads, relatively small voids, such as galleries, shall normally not be deducted except in low diversion structures where such voids are likely to create appreciable effect upon the stability of the structure. The dead loads shall be assumed to act at the center of gravity of the element or equipment under consideration.

For preliminary designs, the unit weights of different construction materials used in the dead load calculations shall be based on standard values. However, for final designs, dead load calculations shall be based on actual data from laboratory tests. Similarly, the weights of gates and appurtenances shall be obtained from their manufacturers.

14.2.2 Headwater and Tailwater Pressures

The hydrostatic pressures due to head and tailwater shall be determined from the hydrology, meteorology and pool regulation studies. The pool levels corresponding to different load conditions shall be selected based on an assessment of their frequency.

14.2.2.1 Non-overflow Section

Hydrostatic pressure on the upstream and downstream faces of the non-overflow section shall be assumed to vary linearly with depth and act normal to the face of the section (Figure 14). The tailwater pressure, adjusted for any retrogression, shall be taken at its full value.

Hydrostatic pressures shall be computed assuming the unit weight of water as 10 kN/m^3. Variations in this value due to temperature changes shall normally be neglected.

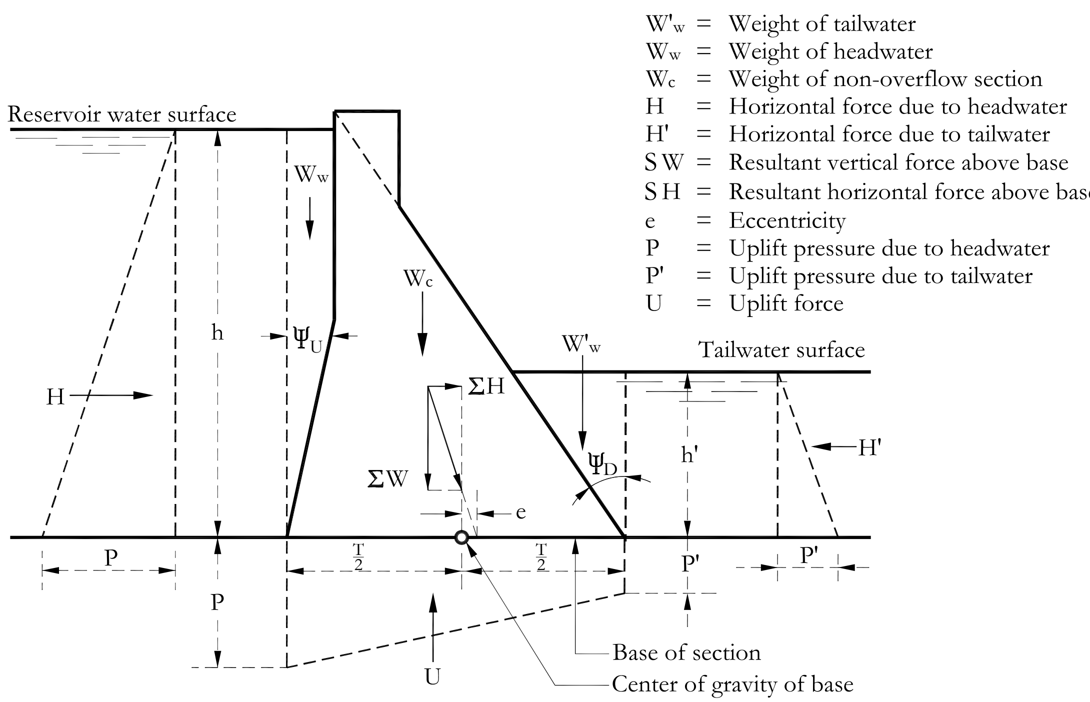


Figure 14: Water pressures acting on non-overflow section (USBR, 1978)

14.2.2.2 Overflow Section

Headwater pressure on gated overflow sections shall be computed assuming the gates to be part of the concrete section. For ungated overflow sections, the lateral pressure distribution due to headwater shall be assumed to be trapezoidal (Figure 15). The vertical pressure of water flowing over the crest shall be neglected in view of the fact that this pressure is greatly reduced due to the water approaching spouting velocity. Because of its high velocity, the stream of water on the downstream face of the overflow section shall also not be considered in the analysis.

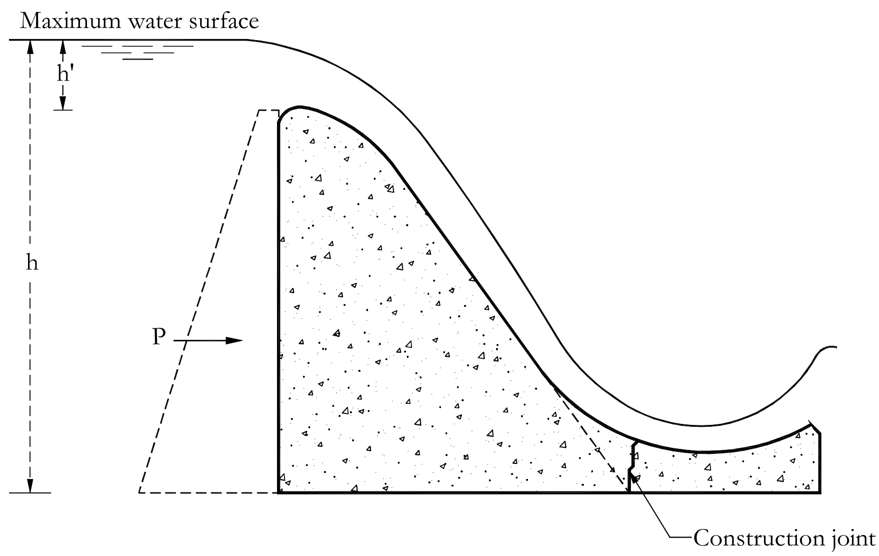


Figure 15: Water pressures acting on overflow sections

For computation of tailwater pressure, the effective tailwater depth shall be suitably reduced to adjust for retrogression. This reduction shall depend on the degree of submergence of the crest, the type of energy dissipation arrangement and the anticipated downstream water profile. For deep flow over the overflow section, the effective depth may be taken equal to 60 percent of the full tailwater depth.

14.2.3 Uplift

Uplift pressure resulting from headwater and tailwater shall be considered to act within the body of the diversion structure, along the interface of the structure and its foundation and within the foundation below the base.

14.2.3.1 Uplift Assumptions

The uplift pressure acting on any failure plane within the diversion structure, along its base or below the base shall be computed based on the following assumptions:

a. Regardless of the overflow conditions, the uplift pressure at the toe of the diversion structure corresponds to the full tailwater depth.
b. Uplift pressures act over the entire area of the failure plane under consideration.
c. Uplift pressures remain unaffected by earthquake loads.

14.2.3.2 Uplift within Diversion Structure

Uplift pressures along failure planes within the body of the concrete diversion structure shall be assumed to vary from 100 percent of the normal headwater at the upstream face to 100 percent of the tailwater or zero, as the case may be, at the downstream face. Where vertical drainage is provided within the diversion structure, the drain effectiveness and uplift assumptions shall follow the guidance provided in Section 14.2.3.3.

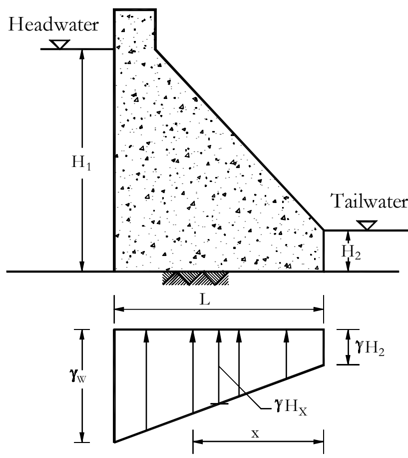
14.2.3.3 Uplift along Base of Structure on Rock Foundation

The uplift pressure along the base of a concrete diversion structure founded on rock shall be estimated considering the following:

a. Provision and effectiveness of measures, such as drains and/or grout curtain and/or aprons, for uplift reduction.
b. Geologic features such as rock permeability, seams, jointing and faulting.

Diversion Structure without Foundation Drains

Where foundation drains are not provided, uplift at the foundation-concrete interface of the diversion structure shall be assumed to vary linearly from the full headwater pressure at the heel of the structure to the full tailwater pressure at its toe. For this case, uplift at any point on or below the foundation shall be determined based on Figure 16.



$$H_X = H_1 + \frac{x}{L}(H_1 - H_2)$$

H_1 = Water depth at heel
H_2 = Water depth at toe

Figure 16: Uplift pressure distribution without foundation drainage (USACE, 1995a)

Diversion Structure with Foundations Drains

Uplift pressures at the base of a diversion structure and in its foundation may be reduced if foundation drains are proposed in the design. The magnitude of reduction in the uplift shall depend on the following factors:

a. Depth, size and spacing of the drains.
b. Geologic character of the foundation.
c. Facility with which the drains can be inspected and maintained.

Reduction in uplift pressure due to foundation drains shall be assumed only if the geologic characteristics of the foundation have been thoroughly investigated and the drainage system has been designed to correct specific deficiencies of the site. Furthermore, uplift reduction shall be assumed for only if the structure has an open drainage system drain with drain outlets accessible for inspection and maintenance. The reduction may also be assumed for closed drainage systems if a monitoring system is installed to verify uplift pressures on a periodic basis.

Where the above conditions are satisfied, the uplift pressure along the base of the diversion structure may be assumed to vary linearly from the undrained pressure head at the heel of the structure to the reduced pressure head at the line of drains to the undrained pressure head at the toe (Figure 17). For determining the reduced pressure head, the effectiveness of the drainage system may be assumed to vary from 25 to 50 percent. Where the line of drains intersects the foundation within a distance of 5 percent of the headwater depth from the upstream face, the uplift may be assumed to vary as a single straight line (Figure 18). If the drainage gallery is located above tailwater level, the pressure at the line of drains shall be determined as though the tailwater level is equal to the gallery elevation.

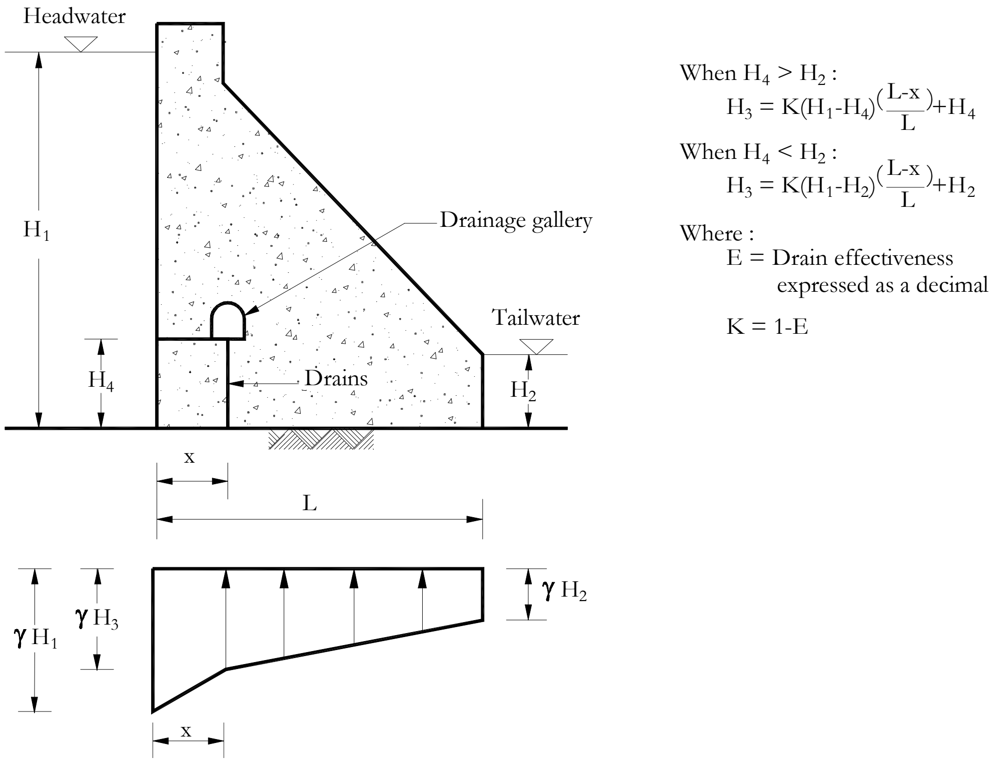


Figure 17: Uplift pressure with drainage gallery (USACE, 1995a)

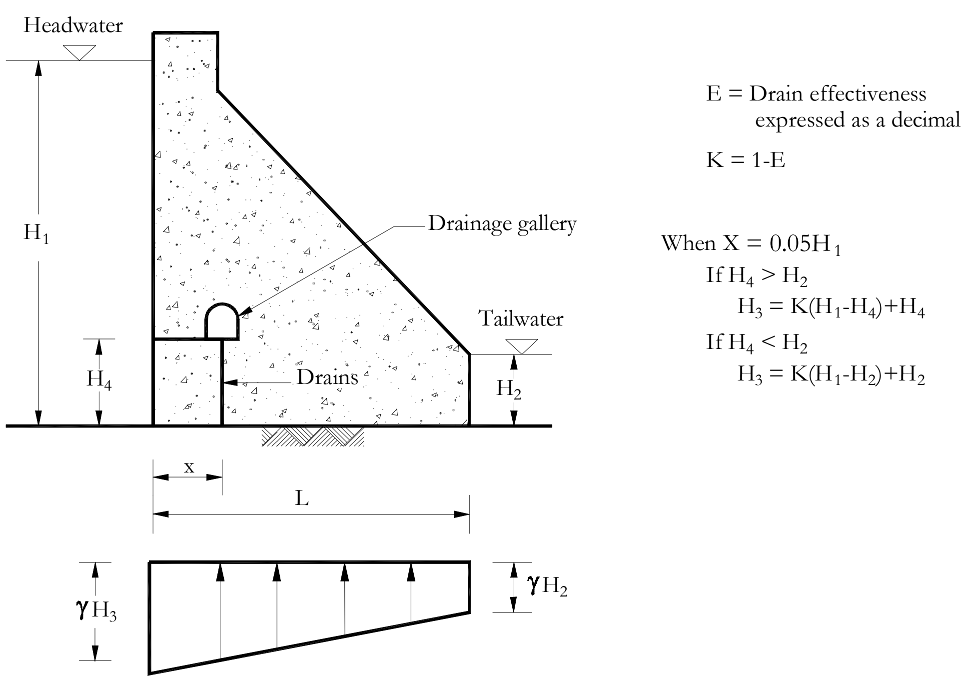
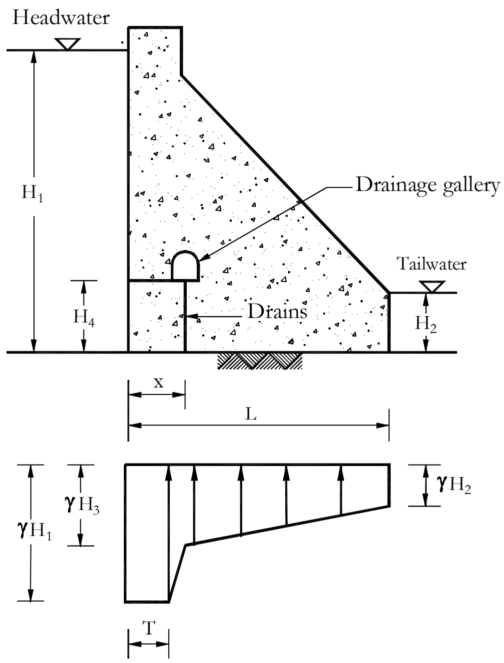


Figure 18: Uplift pressure with foundation drains near upstream face (USACE, 1995a)

Zero Compression Zones

Uplift on any portion of any foundation plane not in compression shall be 100 percent of the hydrostatic head of the adjacent face, except where tension is the result of instantaneous loading resulting from earthquake forces. When the zero compression zone does not extend beyond the location of the drains, the uplift shall be as shown in Figure 19. For the condition where the zero compression zone extends beyond the drains, drain effectiveness shall not be considered (Figure 20).



When $H_4 > H_2$:

$$H_3 = K*[(H_1\text{-}H_2)*\frac{(L\text{-}x)}{(L\text{-}T)}+H_2\text{-}H_4]+H_4$$

When $H_4 < H_2$:

$$H_3 = K*[(H_1\text{-}H_2)*\frac{(L\text{-}x)}{(L\text{-}T)}]+H_2$$

Where :

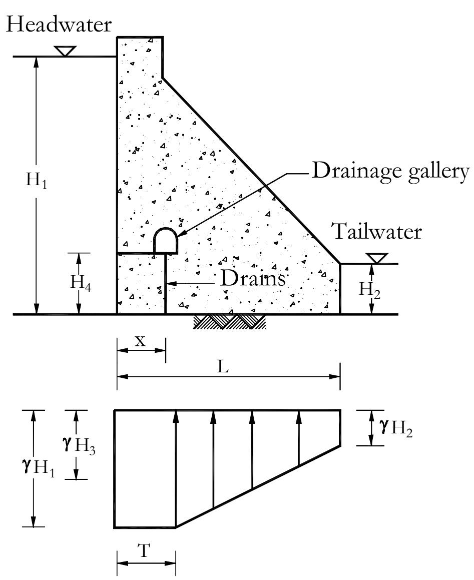
E = Drain effectiveness expressed as a decimal

K = 1-E

T = Zero compression length

T < X

Figure 19: Uplift distribution with zero compression zone not extending beyond drains (USACE, 1995a)



Where :

T = Zero compression length

T < X

Figure 20: Uplift distribution with zero compression zone extending beyond drains (USACE, 1995a)

Impervious or Grouted Foundation

A grouted or a naturally-occurring impervious rock foundation alone shall not be considered sufficient justification to assume an uplift reduction. The reduction shall be allowed only if foundation drains meeting the aforementioned requirements are provided.

Aprons

Reduction in uplift pressure at the heel of the diversion structure due to an upstream apron shall be considered if the joints in the apron and the joints between the diversion structure and apron shall form an impermeable barrier throughout the range of movement expected in the joints due either to loading, differential settlement or contraction due to temperature changes. In the case of downstream aprons, the uplift may be assumed to be limited to that which would float the apron.

Flood Loading

Uplift reductions shall not be based on the assumption that the IDF flood event shall be of such short duration and the foundation permeability so low that the elevated headwater and tailwater pressures are not transmitted under the base of the diversion structure. For design purposes, uplift shall be assumed to vary directly with changes in headwater and tailwater levels.

14.2.3.4 Uplift along Base of Structure on Pervious Foundation

The magnitude of uplift pressures acting upon the base of a diversion structure constructed on pervious foundation shall be estimated using the weighted creep method, the flow net method or the finite element method.

Weighted Creep Method

Using creep theory, the uplift pressure at any point shall be computed from the sum of the seepage potential and position potential.

Flow Net Method

The flow net method shall be used for computing uplift pressures if the actual foundation stratigraphy and boundary conditions can be represented as homogeneous and isotropic. In this method, flow lines and equipotential lines shall be constructed for the subsurface flow, and the uplift pressure shall be computed.

Finite Element Method

Two and three-dimensional finite element ground water modeling may be used to determine the uplift pressure in permeable foundations. This method may be adopted for sites where material anisotropy is likely to have a significant effect on uplift pressures.

14.2.3.5 Uplift within Foundation

Uplift pressure distribution along identified foundation failure planes shall normally assume a uniform head loss along the failure surface. Reduction in uplift pressures along the failure plane may be assumed if the foundation drains penetrate them and are effective.

14.2.4 Nappe Pressures

At small discharges, nappe forces on the overflow section due to steady state hydrodynamic effects may be neglected. However, as the discharge over an overflow section approaches the design discharge, nappe forces with overturning effect on the overflow section, namely sub-atmospheric crest pressures and dynamic bucket pressures, shall be considered in the stability analysis. In particular, sub-atmospheric crest pressures shall be considered for high overflow crests while dynamic bucket pressures shall be taken into account for both high and low overflow sections.

Using this method, the depth of flow at the point under (Figure 21) consideration shall be computed by solving the following relation numerically for the given unit discharge (Brand, 1999)

Eq. 37 $$q = -\sqrt{2g}\sqrt{E - Y - A\sin\phi}(1-\kappa A)\frac{1}{\kappa}\log_e(1-\kappa A)$$

where q is the unit discharge, E is the total energy, Y is the elevation of the point on the overflow section under consideration, A is the depth of flow measured perpendicular to the surface of the overflow section, ϕ is the angle of the outward directed to the overflow section with the horizontal, κ is the curvature of overflow surface at point under consideration taken positive for buckets and negative for crests, L_h is the horizontal length of creep path and L_v is the vertical length of creep path.

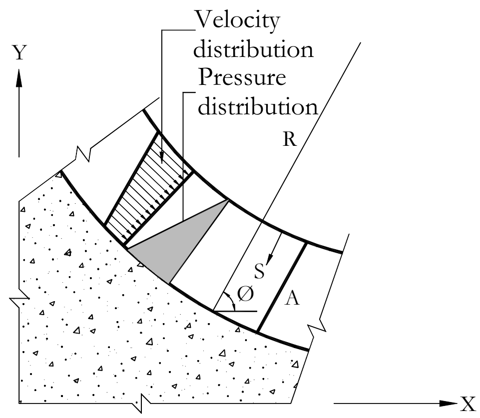


Figure 21: Velocity and pressure exerted by water on overflow section

With A determined, the velocity of flow at the overflow surface shall be found from the equation (Brand, 1999)

Eq. 38 $$V_s = \sqrt{2g}(1-\kappa A)\sqrt{E - Y - A\sin\phi}$$

The pressure head at the overflow surface shall then be determined from the equation (Brand, 1999)

Eq. 39 $$H_s = E - Y - \frac{V_s^2}{2g}$$

14.2.5 Earthquake Forces

The seismic coefficient or the pseudo-static method shall be used for stability analysis of concrete diversion structures under horizontal seismic acceleration. Two types of loadings shall be considered – inertia force due to horizontal acceleration of the diversion structure and hydrodynamic forces resulting from the reaction of the reservoir water against the diversion structure (Figure 22).

14.2.5.1 Inertia Force

The inertia force due to horizontal and vertical acceleration of a section or element of the diversion structure shall be calculated from the equations

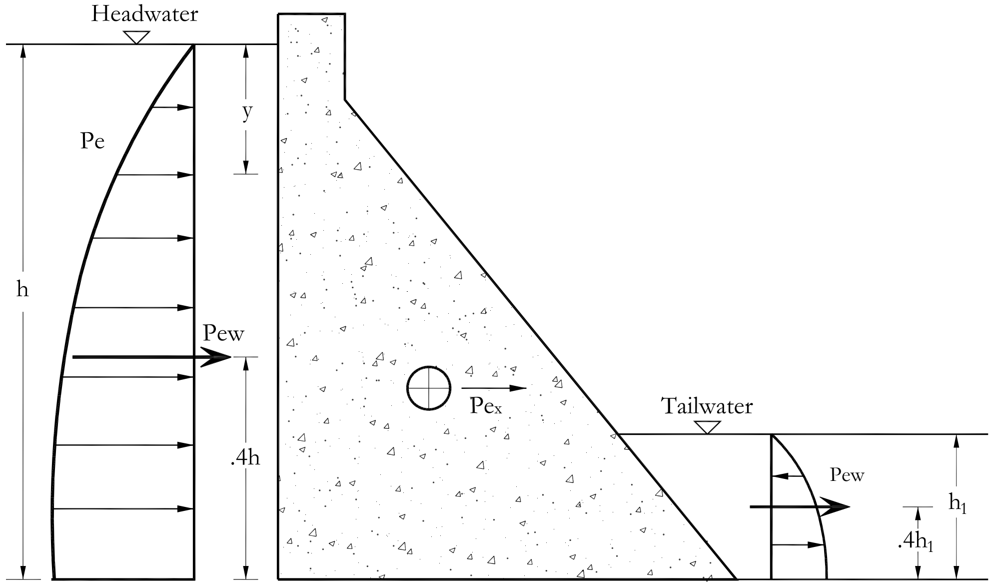


Figure 22: Seismically loaded non-overflow monolith

Eq. 40 $$P_{eh} = \alpha_h W$$

Eq. 41 $$P_{ev} = \alpha_v W$$

where P_{eh} is the horizontal seismic force, P_{ev} is the vertical seismic force, α_h is the horizontal seismic coefficient, α_v is the vertical seismic coefficient and W is the weight of the section or element under consideration.

This force shall be assumed to act at the center of gravity of the section or element under consideration (Figure 22).

The value of α_h shall be based on the design earthquakes and site-specific ground motions determined from seismological evaluation of the site. This value shall be assumed uniform for the total height of the diversion structure and the foundation.

14.2.5.2 Hydrodynamic Force

For diversion structures with vertical or sloping upstream faces, the increase in water pressure due to horizontal earthquake shall be calculated as (USACE, 1995a)

Eq. 42 $$p_d = C_{hy} \alpha_h \gamma_w h$$

where

Eq. 43 $$C_{hy} = \frac{C_m}{2}\left[\frac{y}{h}\left(2 - \frac{y}{h}\right) + \sqrt{\frac{y}{h}\left(2 - \frac{y}{h}\right)}\right]$$

where γ_w is the unit weight of water, h *is* the depth of reservoir, y is the distance measured from water surface and C_m is a coefficient approximately related to the angle of slope θ of the upstream face by

Eq. 44 $$C_m = 0.73\left(\theta / 90°\right)$$

The incremental forces and moments due to pressure p_d shall be approximately taken as

Eq. 45 $$P_d = 0.726\, p_d\, y$$

Eq. 46 $$M = 0.299\, p_d\, y^2$$

where P_d is the total incremental horizontal force above any elevation y and M is the total incremental overturning moment M above any elevation y. The horizontal force P_d shall be assumed to act normal to the face of the diversion structure (Figure 22).

For diversion structures with combination vertical and sloping faces, the procedure to be used for computing hydrodynamic forces shall depend on the relation of the height of the vertical portion to the total height of the diversion structure. If the height of the vertical portion of the upstream face of the structure is equal to or greater the one-half of the total height of the structure, the structure shall be analyzed taking its upstream face as vertical. Otherwise, the hydrodynamic pressure shall be computed for a sloping line connecting the point of intersection of the upstream face of the structure and the reservoir with the point of intersection of the upstream face of the structure with the foundation.

14.2.6 Earth Pressure

Earth pressure due to backfill deposited in the foundation excavation and embankment fills abutting and wrapping around concrete monoliths shall be found based on dry, saturated or submerged unit weights of the material. The state of the lateral earth pressures shall depend on the resulting lateral deformation of the structure. In general, diversion structures shall be designed for active earth pressures if built on soil foundations and for at-rest earth pressures if constructed on rock foundation. The rigidity of the foundation and the character of the backfill, along with the construction sequence, may affect this assumption.

The earth pressure P_{ep} exerted on the diversion structure shall act at one-third of the backfill height and shall be calculated by the relation

Eq. 47 $$P_{ep} = \frac{C}{2}\gamma_f h_f^2$$

where

Eq. 48 $$C = \frac{1}{\cos^2\alpha} \times C_1 \times \left[1 + C_3\left[\frac{\sin(\phi+\delta)}{\cos(\alpha-i)}C_2\right]^{1/2}\right]^{-2}$$

and γ_f is the unit weight of the backfill, h_f is the height of the backfill, ϕ is the angle of friction of the soil, α is the angle which the earth face of the diversion structure makes with the vertical, δ is the angle of friction between the diversion structure and the backfill and i is the slope of the backfill

For active earth pressure,

Eq. 49
$$C_1 = \frac{\cos^2(\phi-\theta-\alpha)}{\cos(\delta+\theta+\alpha)}$$
$$C_2 = \frac{\sin(\phi-\theta-i)}{\cos(\delta+\theta+\alpha)}$$
$$C_3 = +1.0$$

For passive earth pressure,

Eq. 50

$$C_1 = \frac{\cos^2(\phi - \theta + \alpha)}{\cos(\delta + \theta - \alpha)}$$

$$C_2 = \frac{\sin(\phi - \theta + i)}{\cos(\delta + \theta - \alpha)}$$

$$C_3 = -1.0$$

14.2.7 Silt Pressure

In view of the large amount of sediments carried by Nepali rivers, all diversion structures shall be analyzed for silt pressure. The vertical silt pressure shall be estimated using the weight of the silt measured in water, the magnitude of pressure varying directly with depth. In the horizontal direction, the pressure shall be calculated as (JNCD, 1976)

Eq. 51

$$p_s = C_e \gamma_s d$$

where p_s is the silt pressure in the horizontal direction, C_e is the coefficient of silt pressure, generally in the range of 0.40 – 0.60, γ_s is the unit weight of silt deposit measured in water and d is the depth of silt deposit.

For computing the silt pressure, d shall be adopted after careful study of the probable depth of silt deposits within the reservoir during the expected useful life of the diversion structure, The unit weight of silt deposit, γ_s, shall be calculated as

Eq. 52

$$\gamma_s = \gamma_{sa} - (1 - \nu)\gamma_w$$

where γ_{sa} is the apparent unit weight of silt deposit measured in water and ν is the porosity of the deposited silt. Unless specific measurements are available, γ_{sa} shall be taken between 15 and 18 kN/m^3 while ν shall be adopted between 0.30 and 0.45.

In the absence of reliable test data, the horizontal silt and water pressure may assumed to be equivalent to that of a fluid with a mass of 13.6 kN/m^3. Likewise, the vertical silt and water pressure may be determined as if silt and water together have a density of 19.25 kN/m^3.

14.2.8 Wind Pressure

Wind pressure may generally be ignored in the analysis of diversion structures. However, for diversion structure superstructures carrying large crest gates, a wind pressure of 1.5 KPa in any direction, acting on the exposed surface, shall be used in the stability investigations.

14.2.9 Wave pressure

In addition to hydrostatic pressure, the upper portions of concrete diversion structures shall be subject to wave pressure resulting from the impact of waves. The wave pressure diagram shall be approximated by a triangle with its maximum ordinate equal to (IS: 6512 – 1984)

Eq. 53

$$p_w = 0.024 h_w$$

where p_w is the maximum wave pressure in MPa and h_w is the height of wave in m.

The maximum pressure p_w shall occur at a height of 0.125 h_W above the still water level.

The total wave force P_w, in kN, resulting from the wave pressure shall be obtained from the equation

Eq. 54

$$P_w = 20 h_w^2$$

This force shall be applied at a height of 0.375 h_w above the still water level.

14.2.10 Temperature Loads

The effects of temperature change on the stability of concrete diversion structures shall be investigated if the transverse contraction joints between the concrete monoliths are to be grouted or when the operating temperatures are above the closure temperature if joints are to be ungrouted. These effects shall generally include increased twisting between monoliths and additional loading of the abutment. They shall be computed based on the temperature rise and fall as well as the maximum and minimum temperatures expected in the headworks area, taking into consideration factors such as location, orientation, surrounding topography, etc.

14.2.11 Reaction of foundations

Reactions of the foundation shall consist of normal and tangential components that balance the resultant of all applied horizontal and vertical forces, including uplift. For the structure to be in equilibrium, the location of the reactions shall be such that the summation of forces and moments is equal to zero. The distribution of the normal component shall be assumed as linear.

The assumed unit uplift pressure shall be added to the computed unit foundation reaction to the maximum possible unit foundation pressure at any point. For overflow sections, the base width shall generally be found by projecting the downstream slope to the foundation line, and all concrete downstream of this line shall be disregarded.

14.3 Loading Conditions

The loading conditions listed in Table 9 shall generally be used in the stability analysis of concrete diversion structure. Loading conditions such as wave action or unusual loadings that are not indicated shall be included where applicable.

Under earthquake conditions, the selected pool elevation shall be the one judged likely to exist coincident with the selected design earthquake event.

14.4 Stability Calculations

Stability calculations shall be performed to estimate the margin of safety of the structure against overturning, sliding and overstressing. They shall also estimate the stresses in the foundation material to check whether it is overstressed.

14.4.1 Assumptions

Stability analysis shall be performed under the following assumptions:

a. The diversion structure consists of individual transverse vertical elements, each of which carries its load to the foundation without transfer of load from or to adjacent elements.

b. On any horizontal section of the diversion structure, vertical stresses vary linearly from the upstream face to the downstream face.

Where special foundation features and large openings require, stability analysis of the whole block of the diversion structure may be necessary.

Table 9: Basic loading conditions for stability analysis of concrete diversion structures

Load condition	Stage		Description
Usual	A.	Normal operating	• Reservoir elevation at top of closed spillway gates where spillway is provided, and at spillway crest where spillway is ungated • Minimum tailwater

			• Uplift • Ice and silt pressure, if applicable
Unusual	A.	Construction	• Diversion structure structure complete • No headwater or tailwater
	B.	Flood discharge	• Reservoir at standard project flood • Gates at appropriate flood-control openings and tailwater at flood elevation • Tailwater pressure • Uplift • Silt, if applicable • No ice pressure
	C.	Normal operating with OBE	• Operating basis earthquake (OBE) • Horizontal earthquake acceleration in downstream direction • Usual pool elevation • Minimum tailwater • Silt pressure, if applicable • No ice pressure
Extreme	A.	Construction with OBE	• Operating basis earthquake (OBE) • Horizontal earthquake acceleration in upstream direction • No water in reservoir • No headwater or tailwater
	B.	Normal operating with MCE	• Maximum credible earthquake (MCE) • Horizontal earthquake acceleration in downstream direction • Usual pool elevation • Minimum tailwater • Uplift at pre-earthquake level • Silt pressure, if applicable • No ice pressure
	C.	Probable maximum flood	• Reservoir at probable maximum flood (PMF) • All gates open and tailwater at flood elevation • Uplift • Tailwater pressure • Silt pressure, if applicable • No ice pressure

(Based on USACE, 1995a)

14.4.2 Overturning Safety

The overturning stability of the diversion structure shall be found for each loading condition by locating the position of the resultant of the vertical forces and lateral forces along its base. The position of the resultant shall be expressed in terms of its eccentricity defined by

Eq. 55

$$e = \frac{\Sigma M}{\Sigma V}$$

where e is the eccentricity with respect to the centroid of the base of the section under consideration, ΣM is the sum of the moments of the vertical and lateral forces about the downstream toe of the diversion structure and ΣV is the sum of all vertical forces acting on the diversion structure.

14.4.3 Sliding Safety

The sliding stability of a diversion structure shall be based on a limit equilibrium approach. This approach shall consist of determining a factor of safety (FS) against sliding defined as

Eq. 56

$$FS = \frac{T_F}{T} = \frac{N \tan\phi + cL}{T}$$

where T_F is the maximum resisting shear along the slip plane at service conditions, T is the applied shear along the slip plane at service conditions, N is the resultant of forces normal to the assumed sliding plane, ϕ is the angle of internal friction, c is the cohesion on the sliding plane and L is the length of the sliding plane.

Sliding stability analyses shall be based on failure surfaces consisting of a single plane or a combination of planes that are kinematically admissible. In rocks, the failure planes may be predetermined by discontinuities in the foundation. All the potential planes of failure must be defined and analyzed to determine the one with the least FS.

Two-dimensional analysis for sliding stability shall suffice for most cases. However, three-dimensional analysis shall be resorted to if unique three-dimensional geometric features and loads critically affect the sliding stability of a specific structure.

14.4.3.1 Single Plane Analysis

For sliding of any surface within the diversion structure and on single planes of the base, the analysis shall follow the single-plane failure surface analysis in which

Eq. 57

$$FS = \frac{\{W\cos\alpha - U - H\sin\alpha\}\tan\phi + cL}{H\cos\alpha - W\sin\alpha}$$

where W is the total weight of the water, soil, rock and concrete, α is the angle of the sliding plane with the horizontal, U is the uplift pressure against the sliding plane and H is the horizontal force applied to the diversion structure.

14.4.3.2 Multiple Wedge Analysis

Where more than one failure plane exists, sliding stability analysis along the base and within the foundation of the diversion structure shall be conducted using multiple-wedge analysis. For this, the diversion structure and the foundation material acting on it shall be assumed to act as a system of wedges consisting of a structural wedge and driving and resisting wedges, and the sliding factor of safety required to bring the sliding mass into a state of horizontal equilibrium along the given set of slip planes shall be computed, depending upon the category of hazard.

14.4.4 Overstressing

The total stress on any plane in the diversion structure or its foundation shall be calculated by combining the axial and bending stresses at the plane due to the total vertical and lateral loads acting on or above the plane. This shall be achieved through the relation

Eq. 58

$$\sigma = \frac{N}{L}\left(1 \pm \frac{6e}{L}\right)$$

where σ is the total stress on the edges of the plane under consideration, N is the resultant of forces normal to the assumed sliding plane, e is the eccentricity computed using Eq. 55 and L is the length of the sliding plane.

14.5 Stability Criteria

The stability criteria for concrete diversion structures for different load conditions shall be as indicated in Table 10.

Table 10: Stability criteria for concrete diversion structures

Load condition	Resultant location at base	Minimum sliding FS	Foundation bearing pressure	Concrete compressive stresses	Concrete tensile stresses
Usual	Middle third	2.0	≤ allowable	$0.3\ f_c^{'}$	0
Unusual	Middle half	1.7	≤ allowable	$0.5\ f_c^{'}$	$0.6\ f_c^{'2/3}$
Extreme	Within base	1.3	≤ 1.33 x allowable	$0.9\ f_c^{'}$	$1.5\ f_c^{'2/3}$

(Source: USACE, 1995a)

In the table, $f_c^{'}$ is the one year unconfined cylindrical compressive strength of concrete. Concrete allowable stresses are for static loading conditions.

15. STRESS ANALYSIS

Stress analysis of diversion structures shall be performed to determine the magnitude and distribution of stresses throughout the diversion structure under static and dynamic loads and to investigate the structural adequacy of the substructure and foundation. The analysis shall be carried out for the various load conditions discussed in Section 14.2.

15.1 Static Analysis

Static stress analysis of a diversion structure shall be conducted using approximate methods or the finite element method. The analysis technique adopted shall depend on the degree of refinement required for the particular level of design, the complexity of the geometry of the structure and its loading patterns as well as the nature of the foundation material.

15.1.1 Approximate Techniques

Approximate stress analysis shall generally be performed for preliminary design of diversion structures. These analyses shall be carried out using the gravity method for two-dimensional problems and the trial load twist method for three-dimensional problems.

15.1.1.1 Gravity Method

The gravity method shall be used for analysis of diversion structures in which the transverse contraction joints are neither keyed nor grouted. The analysis shall be performed under the following assumptions:

a. The diversion structure consists of a series of vertical cantilever elements.
b. The concrete in the diversion structure is homogeneous, isotropic and uniformly elastic.
c. All loads are transmitted to the foundation through cantilever action of the diversion structure without support from adjacent monoliths.
d. Normal stresses are distributed linearly on horizontal planes.

15.1.1.2 Trial-load Twist Method

The trial load twist method shall be used for the three-dimensional stress analysis of straight diversion structures with keyed transverse joints. It shall be employed in conditions where the torsional moments resulting from the interaction between adjacent vertical cantilevers materially affects the water load distribution between them. Such conditions may arise in:

a. Diversion structures located in a narrow canyon with steep sloping walls.
b. Abutment regions of diversion structures in wide and flat canyons where the length of the cantilever changes rapidly.
c. Region of sharp break in the cross-canyon profile which results in an abrupt change in the length of cantilevers in that region.
d. Regions where the foundation stratigraphy changes.

15.1.2 Finite Element Method

The finite element method shall ordinarily be used for the feature and final design stages of the diversion structure if a more exact investigation of stress is required. Its use shall also be considered for diversion structures that are curved in plan, have unusual configurations or are founded on complicated foundations involving various materials, weak joints on seams and fracturing.

Linear elastic finite element analyses shall normally suffice for diversion structures. Where required, nonlinear finite element analyses that account for interaction of the diversion structure and the foundation may also be conducted. The method may also be used for modeling concrete thermal behavior in conjunction with other loads.

15.1.2.1 Two-dimensional Analysis

Two-dimensional finite element analysis shall normally be adequate for concrete diversion structures. It shall particularly be suitable for analyzing sections of structures where plane strain or plane strains exist. Three-dimensional effects may also be approximated by making a two-dimensional analysis in more than one plane. However, while interpreting the results of these analyses, the limitations of the approximations involved in representing an actual three-dimensional structure with a two-dimensional model shall be borne in mind.

15.1.2.2 Three-dimensional Analysis

Three-dimensional finite element modeling of diversion structures shall be performed when two-dimensional approximations may not represent the true behavior of the structure and its supporting foundation and abutments. Such conditions may arise when the geometry of the problem is such that the behavior of the structure depends upon stress distribution parallel to its axis, as in a structure curved in plan, a structure located in a narrow canyon or when the cross section of the structure or its loading is not uniform.

In view of the large cost and effort involved in three-dimensional modeling, such modeling shall be aimed at grasping the global behavior of the structure. The local behavior of the structure at any particular section may then be studied based on the results of the three-dimensional analysis.

15.1.2.3 Interpretation of Results

The results of the finite element analysis of diversion structures shall be used after their validation through equilibrium and compatibility checks. This shall be done along planes of importance by checking reactions against applied loads and displacements against applied boundary conditions

15.2 Dynamic Analysis

Dynamic stress analysis of a diversion structure shall be conducted to determine its response based on the characteristics of the structure and the nature of the earthquake loading. This analysis shall be performed in the following situations:

- The diversion structure is 30 m or more in height and the peak ground acceleration (PGA) at the site is greater than 0.2 g for the MCE;
- The diversion structure is less than 30 m high and the PGA at the site is greater than 0.4 g for the MCE;
- The diversion structure has gated spillway monoliths, wide roadways, intake structures or other monoliths of unusual shape or geometry.

15.2.1 Dynamic Methods of Stress Analysis

Dynamic methods for stress analysis of diversion structures shall be based on the modal analysis technique. These techniques include a simplified response spectrum method and the finite element methods using either a response spectrum or acceleration-time records for the dynamic input. A dynamic analysis shall begin with the response spectrum method and progress to more refined methods if needed. A time-history analysis shall be used when yielding (cracking) of the diversion structure is indicated by a response spectrum analysis.

15.2.1.1 Simplified Response Spectrum Method

The simplified response spectrum method shall be used to compute the maximum linear response of a non-overflow section in its fundamental mode of vibration due to the horizontal component of ground motion. It may also be used for an ungated overflow monolith that has a section similar to a non-overflow monolith as well.

Using this method, the diversion structure shall be modeled as an elastic mass that is fully restrained on a rigid foundation. Hydrodynamic effects shall be modeled as an added mass of water moving with the structure, the amount of the added water mass depending on the fundamental frequency of vibration and mode shape of the structure and the effects of interaction between the structure and reservoir. Seismic loading shall be computed directly from the spectral acceleration obtained from the design earthquake response spectrum and from the dynamic properties of the structural system.

15.2.1.2 Finite Element Methods

Finite element methods for dynamic stress analysis shall be used to model the response of the higher modes of vibrations of diversion structures.

Modeling Approach

Finite element models for dynamic analysis shall accurately model the structure's geometry and, in particular, shall include voids and any lumped masses which might affect the vibration mode shapes of the structure. Because variations in modeling parameters and uncertainties in material properties significantly affect stress distributions in the structure, a number of analyses shall be performed to test the sensitivity of the dynamic behavior to various combinations of properties and assumptions. The ultimate strengths of the materials may be used in earthquake analyses using dynamic methods.

Finite Element Response Spectrum Method

The finite element response spectrum method shall be used to model the dynamic response of linear two- and three-dimensional diversion structures. It shall be used for monoliths that cannot be modeled two-dimensionally or if the maximum tensile stress computed from the simplified response spectrum method exceeds 15 percent of the unconfined compressive strength of the concrete.

In this analysis, hydrodynamic effects shall be modeled as an added mass of water moving with the diversion structure using Westergaard's formula. The foundation shall be modeled as discrete elements or a half space.

The finite element analysis shall be used to compute the natural frequencies of vibration and corresponding mode shapes for specified modes. Earthquake loading shall be computed from earthquake response spectra for each mode of vibration induced by the horizontal and vertical components of ground motion. These modal responses shall be combined using the complete quadratic combination method, or the square root of the sum of squares method for two-dimensional structures, to obtain an estimate of the maximum total response. Normal stresses shall then be computed by a static analysis of the structure using the seismic loading as an equivalent static load.

Finite Element Acceleration – Time History Method

An acceleration-time history analysis shall be performed if the variation of stresses with time is required to evaluate the extent and duration of a highly stressed condition. It shall also be used to determine the number of cycles of nonlinear behavior, the magnitude of excursion into the nonlinear range and the time the structure remains nonlinear.

In this method of analysis, hydrodynamic effects may be modeled using the wave equation. Compressibility of water and structural deformation effects shall be included in computing the hydrodynamic pressures. The dynamic input shall be in the form of an acceleration-time record.

The acceleration-time history method shall yield the natural frequencies of vibration and corresponding mode shapes for specified modes. The response of each mode, in the form of equivalent lateral loads, shall be calculated for the entire duration of the earthquake acceleration-time record starting with initial conditions, taking a small time interval, and computing the response at the end of each time interval. The modal responses shall be added for each time interval to yield the total response. Thereafter, normal and principal stresses shall be computed by a static analysis for each time interval.

15.2.2 Performance Criteria for Response to Site-Dependent Earthquakes

Concrete diversion structures shall be capable of surviving the controlling MCE without a catastrophic failure that would lead to loss of life or significant damage to property. Inelastic behavior with associated damage shall be permissible under the MCE. Furthermore, they shall be capable of resisting the controlling OBE within the elastic range, remain operational and not require extensive repairs.

16. PIERS

On uncontrolled overflow sections, piers shall primarily be provided as supports for bridges. On controlled spillways, piers shall be used to divide the overflow section into a number of bays so that the flow over the overflow section can be controlled with gates. They shall also contain the anchorage or slots for the crest gates and may support fixed hoists for the gates.

Piers shall be erected over the crest profile and shall generally be located in the middle of the monolith. The spacing between piers shall depend on the size of the gates; however, it shall be sufficiently large to reduce the risk of timber and ice blocking the overflow bays.

16.1 Proportioning

The dimensions of the pier shall be fixed based on considerations of flow conditions, loads and requirements of appurtenances supported by the piers.

16.1.1 Width of Pier

The width of the piers shall be fixed considering the following factors:

a. Forces and moments transferred by the pier to the overflow section.
b. Size of gates.
c. Minimum thickness required at the block-outs for the crest gates and stop log grooves.
d. Mass of the pier required for counteracting uplift pressure.

The width of the pier shall generally not be less than 2.5 to 3 m. Furthermore, the separation between block-outs on either face of the pier shall not be less than 600 mm.

16.1.2 Length of Pier

The length of the piers shall be fixed based on the following considerations:

a. Minimum requirements of bridge and hoist bridge provided for operation of the gates and stop logs.
b. Space required for housing instruments, if any.
c. Width of grooves for crest gates and stop logs.
d. Space for storage of stop logs.
e. Prevention of cross flows.

16.1.3 Height of Pier

On the upstream side, the pier shall generally be constructed above the pond level affluxed HFL with adequate free board. The height shall be fixed as per requirement of the mass of the pier in counteracting uplift pressure. It shall also be such that about 1 m of the gate still remains within the gate groove when fully raised position above the affluxed HFL/pond level.

On the downstream side, the piers shall generally be constructed at least one meter above the high flood level up to a distance of 1 to 2 m, as necessary, beyond the end of the bridges and instrumentation platform, if any. Thereafter, the height could be reduced according to low flood levels on the downstream side. In portions where bridges are provided, the height of the piers shall be fixed such that the bearings of the bridges are not hit by floating debris during high floods.

In the main gate portion, the height of the pier shall be fixed such that the bottom of the gate is at least 1 m clear of the affluxed HFL. For reducing the loads and moments due to earthquake, however, the top level of the pier could be restricted to the top level of the abutments and steel trestles provided over the piers for housing the hoist bridges.

16.1.4 Pier Cap

Properly designed pier caps shall be provided under bridges. Their thickness shall not be less than 300 mm.

16.2 Structural Design

The piers shall be designed to resist the most critical combination of axial, bending, shear and torsional stresses developed under the worst condition of loading.

16.2.1 Load Cases

The loads to be taken into consideration for the design of piers shall be as follows:

a. Self weight of pier.
b. Weight of bridges on the pier
c. Uplift pressure on piers.
d. Weight of hoisting equipment.
e. Upward water pressure on gates.

f. Weight of gate to be resisted by pier.
g. Reaction due to live load on the bridge including impact.
h. Crane loads, if provided.
i. Transverse water pressure on the pier.
j. Force due to braking effect of vehicles.
k. Frictional resistance at the bearing of the road bridge.
l. Pin reaction in transverse direction due to water pressure on radial gate with inclined arms.
m. Wind load.
n. Earthquake loads, including hydrodynamic forces.
o. Longitudinal static water pressure on the pier.
p. Forces due to water current.

16.2.2 Load Conditions

The pier shall be designed for the most critical combination of loads listed in Section 16.2.1. Since each pier supports a gate on each side, the following pier loading conditions (Table 11) shall be investigated.

Cases 1 and 3 shall be considered to calculate the maximum horizontal shear normal to the axis of the diversion structure and the largest overturning moment in the downstream direction. Case 2 shall be used to determine the lateral bending moment due to the water flowing through the open gate and the torsional shear in the horizontal plane introduced by the reaction of the closed gate acting on one side of the pier.

Table 11: Load cases for design of piers

Case	Description
1	Both gates closed and water at the top of gates.
2	One gate closed and the other gate wide open, with water at the top of the closed gate.
3	One gate closed and the other open, with stop logs in place and water at the top of the closed gate.

16.2.3 Block-out Zones

Block-out zones shall be designed to withstand the stresses due to the worst combination of forces and moments listed in Section 16.2.2 and the differential hydrostatic pressure with gate closed on one side and stop logs dropped on the other adjacent side. Adequate dowel bars and secondary reinforcement shall be provided to effectively bond the second stage concrete to the main pier concrete.

16.2.4 Reinforcement Design

The pier reinforcement shall be designed assuming the pier as an eccentric column for the vertical and horizontal loads and bending moments. It shall be properly anchored in the overflow section. The minimum pier reinforcement shall not be less than 25 mm diameter bars spaced at 300 mm centers.

17. DIVIDE WALLS

Divide walls shall be provided in an overflow section for any one or more of the functions:

a. To separate bays having different types of energy dissipation arrangements.
b. To separate bays having the same type of energy dissipation arrangement which have different levels or parameters because of geological or other considerations.

c. To allow for unsymmetrical operation of crest gates in order to minimize cross or return flows, eddies, etc.

17.1 Proportioning

In order to minimize disturbance to the flow, divide walls shall normally be provided in line with the crest piers. Their sections shall be made as thin as possible.

The height of divide walls shall be fixed based on the requirements for clear separation of flow. Where clear separation is required, the walls shall be provided with sufficient freeboard over the maximum tailwater level. Submerged walls shall be provided in situations where clear separation of flow is required at lower discharges only. In this case, the extent of submergence shall be decided by model tests.

The divide wall shall usually extend up to the downstream end of the energy dissipators. However, its length may be determined based on model studies.

17.2 Structural Design

Structural design of a divide wall shall be performed to determine the reinforcement needed for resisting stresses developed in them under the worst condition of loading.

17.2.1 Design Loads

The following loads shall be considered for the design of the divide walls:

a. Dead loads;
b. Water pressure including hydrodynamic pressures;
c. Uplift pressures, and
d. Earthquake forces.

17.2.1.1 Computation of Loads

Dead load shall consist of the self weight of the divide wall. Any superimposed load on the walls may also be included in the computation of the dead load.

The uplift pressure shall be assumed to act over 100 percent of the base area of the wall. It shall be assumed to vary uniformly along the base width. It may be safe to assume that uplift pressures are not affected by an earthquake.

Estimates of hydro-dynamic pressures due to turbulence and surges shall be based on model studies. However, till results of such tests are available, estimates of these pressures may be made based on simplified assumptions.

17.2.2 Load Conditions

The divide wall shall be designed for the load conditions listed in Table 12.

18. CONTRACTION JOINTS

Transverse contraction (monolith) joints shall be provided in concrete diversion structures for the following purposes:

a. Prevent the formation of haphazard ragged cracks resulting from volume changes that cannot be prevented in the mass concrete.
b. Divide the structure into convenient sized monoliths to permit convenient and systematic construction.

Table 12: Loading conditions for divide walls

Condition	Loads

Unsymmetrical spillway operation	• Dead loads • Hydrostatic and hydrodynamic loads due to unsymmetric flow in the energy dissipator, including centrifugal forces in the case of flip buckets • Corresponding full uplift
Spillway not discharging	• Dead loads • Hydrostatic and hydrodynamic loads on only one side of wall • Corresponding full uplift • Earthquake forces

18.1 Arrangement of Joints

The joints shall be vertical and normal to the axis of the diversion structure, and they shall extend continuously through its section. In order to assure freedom of volumetric change of monoliths, they joints shall be constructed so that bonding does not exist between adjacent monoliths. Reinforcing should not extend through the joints. Above the minimum pool level, the joints shall be chamfered at the faces of the diversion structure for appearance and for minimizing spalling.

18.2 Location and Spacing

The transverse contraction joints shall generally be spaced uniformly across the axis of the diversion structure 15 to 25 m apart. The location and spacing of the joints shall be decided considering the physical features of the site, details of appurtenant structures, results of temperature studies, placement rates and methods and the probable capacity of the concrete mixing plant. Factors such as abrupt discontinuities in the profile of the diversion structure, material changes, defects in the foundation and the location of other features shall also be considered in deciding these parameters. In the overflow section, factors such as gate, pier size and other requirements shall also be taken into account for determining joint spacing.

18.3 Shear Keys

Where foundation conditions of the diversion structure are such that undesirable differential settlement or displacement may occur, shear keys may be formed in the contraction joints (Figure 23). These joints may be formed vertically, horizontally or as a combination of both depending on the direction of the expected displacement. The keys may be rectangular or circular in shape.

The shear keys shall be capable of withstanding the shear stresses due to transfer of loads from adjacent monoliths. Their structural design shall be performed based on the concept of shear friction.

18.4 Waterstops

A double line of waterstops shall be provided near the upstream face at contraction joints to prevent leakage of water through them (Figure 24). Normally, polyvinyl chloride (PVC) waterstops shall be used for this purpose.

The waterstops shall be grouted 450 to 600 mm into the foundation or sealed to the cutoff system and shall terminate near the top of the diversion structure. For gated overflow sections, the tops of the waterstops shall terminate near the crest of the ogee. A single line of waterstops shall also be placed around all galleries and other openings crossing monolith joints.

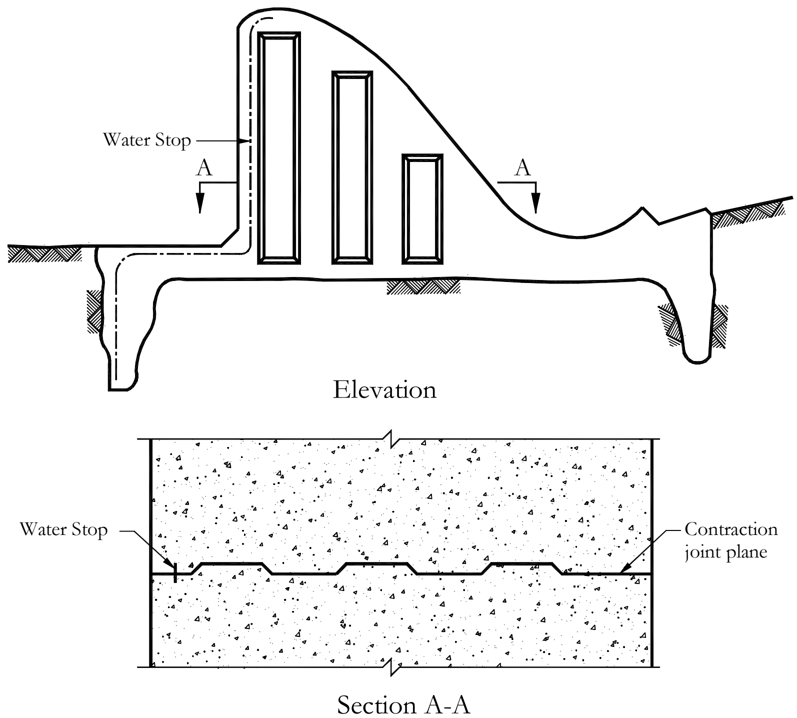


Figure 23: Typical contraction joint shear keys

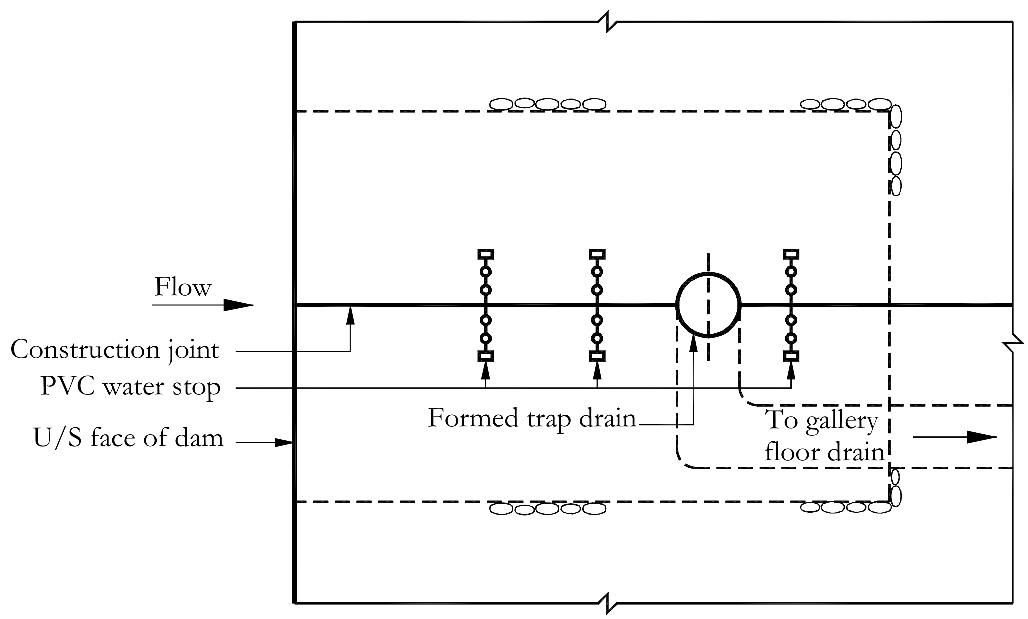


Figure 24: Typical waterstop installation

18.5 Formed Drains

A 150 to 200 mm diameter formed drain shall generally be provided between the two water stops (Figure 24). In the non-overflow monolith joints, the drains shall extend from the maximum pool elevation and terminate at about the level of, and drain into, the gutter in the grouting and drainage gallery. In the overflow monolith joints, the drains shall extend from the gate sill to the grouting and drainage gallery.

19. GALLERIES AND OTHER OPENINGS

A system of galleries, adits, chambers and shafts shall usually be provided within the body of the diversion structure for the following purposes:

a. To provide drainage way for water seeping through the upstream face of the diversion structure and from the foundations.
b. To furnish means of access and space for drilling and grouting.
c. To provide means for installation, operation and maintenance of accessories and utilities in the diversion structure.
d. To provide access to the interior of the diversion structure for inspection.

19.1 Types of Galleries and Openings

To meet the above-listed purposes, the galleries and openings discussed in the following sections shall be provided within the diversion structure. These facilities shall be arranged in the diversion structure considering their functional usefulness and efficiency and their location with respect to maintaining the structural integrity.

19.1.1 Grouting and Drainage Gallery

The grouting and drainage gallery shall extend the full length of the diversion structure for grouting the foundation cutoff and for collecting seepage from foundation drainage holes and the interior drainage holes. Generally, it shall be provided for diversion structures whose height above the normal foundation level exceeds 10 m.

The gallery shall be located near the upstream face and as near the rock surface as feasible to provide the maximum reduction in overall uplift. A minimum distance of 1.5 m shall be maintained between the foundation surface and the gallery floor. The minimum distance between the upstream face and the gallery upstream wall shall be the greater of 3 m or 5 percent of the pool head measured from the NWL to the foundation level. The minimum size of the gallery shall be 1.5 m wide by 2.25 m high; however, a larger dimension, about 2 m wide by 2.5 m wide, shall be preferred to facilitate drilling and grouting operations.

19.1.2 Gate Chambers and Access Gallery

Gate chambers shall be located directly over the service and emergency sluice gates. These chambers shall be sized to accommodate the gate hoists along with related mechanical and electrical equipment and shall provide adequate clearances for maintenance. Access galleries shall be of sufficient size to permit passage of the largest component of the gates and hoists and equipment required for maintenance.

19.1.3 Inspection Gallery

Inspection galleries shall be provided in a diversion structure for access to the interior mass

of the structure and to study the structural behavior of the structure. Grouting and drainage galleries may also be used for this purpose. In overflow sections with height greater than 25 m, inspection galleries may be provided at different levels above the grouting and drainage gallery. However, the top of the gallery at the highest level shall not be located within 7 m of the crest level of the overflow section. The size of the gallery shall be 1.5 m wide by 2.5 m high.

19.1.4 Instrumentation Gallery

The number and location of instrumentation galleries shall depend upon the extent of instrumentation provided in the diversion structure. The galleries shall generally be aligned perpendicular to the axis of the diversion structure. They shall generally have a rectangular cross-section 1.5 m wide and 2.25 m high.

In diversion structures of low height where separate instrumentation galleries cannot be provided, instrumentation may be installed in a suitable space in the grouting and drainage gallery.

19.1.5 Sump Well and Pump Chamber

Sump wells shall be provided in the deepest location. Their number and size shall depend upon the quantity of water seeping through the foundations and the body of the diversion structure.

Chambers for housing pumps for pumping off the water collected in the sump wells shall be located adjacent to an inspection gallery. It may also be located in an area that can be easily assessed in the event of the drainage gallery being flooded.

19.1.6 Shafts

Shafts for staircases and elevators shall be provided for access to galleries from the top of the diversion structure. They shall be located in the non-overflow section, generally near the interface of the non-overflow section with the overflow section.

19.2 Gallery Details

Galleries may be usually arranged as a series of horizontal runs and stair flights or ramps with slopes preferably not exceeding 10º with the horizontal. The stairs and ramps shall be provided with safety treads or a non-slip aggregate finish. Metal treads may be preferable where it is probable that equipment shall be skidded up or down the steps. Where practicable, the width of tread and height of riser shall be uniform throughout all flights of stairs and shall never change in any one flight. Normally, steps shall not cross contraction joints between adjacent monoliths.

All galleries shall be provided with gutters to carry away seepage water which gets collected into the gallery. On horizontal runs, the depth of the gutter may vary from 225 to 375 mm to provide a drainage slope. The drainage slope shall not be flatter than 1 in 1,000.

19.3 Structural Design

Structural design of galleries and other openings shall aim at strengthening the concrete around them to resist the high compressive and tensile stresses developed in these regions. This shall be achieved through the analysis and design procedures discussed in the following sections.

19.3.1 Stress Analysis

Stress analysis shall be performed to determine the stress distribution around galleries and openings. For this purpose, a two-dimensional analysis assuming plane stress or plane strain may generally suffice. However, three-dimensional analysis may be required at intersections of two or more openings.

For the stress analysis, the overall stress field prevailing at the center of the gallery shall be determined by approximate methods, such as the gravity method (Section 15.1.1.1), or the finite element method. The stress distribution around the opening under the overall stress field may then be found approximately from closed form analytical solution for circular or elliptical openings or from stress coefficients developed for rectangular openings of various width and height. Finite element analysis may be conducted for openings that have complex forms and load conditions. This analysis may be needed for openings located close to the face of the diversion structure or near the face of a block, for multiple openings in on plane and for intersections.

19.3.2 Load Conditions

Galleries and openings shall be designed for the overall stress field resulting from the most adverse combinations of loads acting on the diversion structure. For this purpose, the load cases discussed in Section 14.2 and the load conditions listed in Section 14.3 shall be used; however, uplift shall not be considered.

19.3.3 Reinforcement

Reinforcement shall be designed for the total tensile force across the plane considered for design. This force shall be determined by integrating the tension stress on the plane. The area of steel reinforcement shall be calculated by dividing the tensile force by the allowable tensile stress of the reinforcement.

Reinforcement provided for resisting tensile stresses shall consist of horizontal and vertical bars on the periphery and diagonal bars at corners. These bars shall be anchored in a zone of compression. The spacing of bars shall generally not be less than 150 mm centers and not greater than 300 mm centers. The minimum clear cover to reinforcement shall be 150 mm. The minimum diameter of reinforcement bars shall be 16 mm for main reinforcement and 12 mm for distribution reinforcement.

20. BRIDGES ACROSS OVERFLOW SECTIONS

Bridges shall be provided across overflow sections to furnish access for pedestrian and vehicular traffic between the non-overflow sections, to provide access or support for the operating machinery for the crest gates; or, usually, to serve both purposes. In the case of an ungated spillway and in the absence of vehicular traffic, access between the non-overflow sections may be provided by a small access bridge or by stair shafts and a gallery beneath the spillway crest.

20.1 Design Criteria

The class of bridge design loading shall normally not be less than Indian Road Congress (IRC) Class AA loading. Special loadings required for operation and maintenance functions and those that the bridge is subjected to during construction shall be taken into account, including provisions for any heavy concentrated loads. Heavy loadings for consideration shall include those due to equipment transported during construction, mobile cranes used for maintenance and gantry cranes used to operate gates and to install stop logs for the overflow section. If the structure carries a highway, the design shall usually conform to the standard specification for highway bridges in Nepal.

20.2 Materials

Materials used in the design and construction of the bridge shall be selected on the basis of life cycle costs and functional requirements. Floors, curbs and parapets shall be reinforced concrete. Beams and girders may be structural steel, precast or cast-in-place reinforced concrete or prestressed concrete.

PART 2C – ENERGY DISSIPATORS

TABLE OF CONTENTS

Part

2C

Energy Dissipators

1. PURPOSE

Part 2C of the *Design Guidelines for Headworks of Hydropower Projects* provides guidance for the selection and design of appropriate energy dissipators for the overflow sections of diversion structures of headworks of run-of-river hydropower projects in Nepal. The guidelines are intended to ensure safe and economical design of these structures with due consideration of relevant issues, particularly those arising from conditions typical to Nepal.

2. SCOPE

The guidelines cover the design of energy dissipators deemed suitable for overflow sections of run-of-river hydropower projects in Nepal. These dissipators include the hydraulic jump type stilling basins, roller buckets and flip buckets.

The guidelines discuss the design philosophy and principles of the various energy dissipators and provide guidance on their selection in context of typical Nepali conditions. They deal with the hydraulic design of the dissipators and their appurtenances, including training walls, divide walls and downstream protection works. They also provide guidance on the structural analysis and design of these structures.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Baffle blocks Blocks installed on the basin floor between chute blocks and end sill.

Bucket A curved surface provided at the toe of an overflow section to deflect the flow horizontally.

Cavitation Hydrodynamic process occurring on the surface of structure.

Chute blocks Triangular reinforced concrete blocks provided at the toe of the downstream glacis for energy dissipation.

Conjugate depths	Water depths at the beginning and the end of the hydraulic jump.
Crest	Top of hydraulic structure (dam or weir) through which flow of water takes place.
Divide wall	Wall usually constructed at right angles to the axis of the weir or barrage and generally extending beyond the main structure to separate the undersluices, river sluices and spillways into independent units for facilitating regulation.
Drainage	Safe removal of excess seepage water below an energy dissipation structure or from behind a training wall.
End sill	Vertical, stepped, sloped or dentated wall constructed at the downstream end of the stilling basin.
Filter	Layer or a combination of layers of graded pervious materials designed and placed to provide drainage and yet prevent the movement of soil particles with seepage water.
Flip bucket	Bucket which enables the high velocity jet of water to be thrown into the air.
Freeboard	Marginal height above the full supply level.
Froude number	Dimensionless number characterizing the inertial and gravitational forces in an open channel flow.
Glacis	Sloping portion of the floor upstream and downstream of the crest of the river diversion structure.
Hydraulic jump	Sudden and usually turbulent passage of water from low level below critical depth to high level above critical depth during which flow velocity passes from supercritical to sub-critical.
Hydraulic jump type stilling basin	A basin in which dissipation of energy is accomplished basically by hydraulic jump which may be stabilized using chute blocks, basin blocks, end sill, etc.
Length of hydraulic jump	Distance from the beginning of the jump to a point downstream where either the high velocity jet begins to leave the floor or to a point on the surface immediately downstream of the roller, whichever is the longer.
Length of stilling basin	Dimension of the basin in the direction of flow.
Sequent depth	Depth of hydraulic jump in the stilling basin
Slotted roller bucket	A bucket type energy dissipator in which the lip wall is made up of alternate teeth and slots and below which a sloping apron is provided.
Solid roller bucket	A bucket type energy dissipator which consists of a bucket like apron with a concave profile of considerable radius and a lip which deflects the high velocity flow away from the stream bed.
Stilling basin	A short length of paved portion below an overflow section in which all or part of the surplus energy of flowing water is dissipated, and the water is discharged into the downstream channel.
Stilling basin appurtenances	Structures, such as blocks, sills and baffles, installed in the stilling basin to help improve its performance by increasing turbulence and obtaining desired flow conditions downstream.
Tailwater elevation	Elevation of the water surface downstream from a weir or barrage.

Toe of dam	Location where the downstream face of the weir or barrage intersects the foundation.
Training walls	Walls built at the sides of stilling basin or bucket with top set at an elevation so that the maximum tailwater for the design discharge is contained with sufficient freeboard considering spray and air entrainment.
Width of stilling basin	Dimension of the basin perpendicular to the direction of main flow.
Wing walls	Walls constructed at the end of training walls of the basin to prevent them from being undermined.

4. DESIGN OBJECTIVE

Energy dissipators shall be designed to return the high velocity discharge passing over the overflow section of a river diversion structure to the downstream river bed without serious scour or erosion of the toe of the diversion structure or damage to adjacent structures. This objective shall be attained by reducing the high velocity flow to a velocity low enough to minimize erosion of natural river bed material.

5. SCOPE OF DESIGN

The design objective for energy dissipators shall be attained through the following activities:

a. Selection of the most suitable energy dissipation system for the overflow section.
b. Hydraulic design of the selected energy dissipator to establish its physical parameters.
c. Structural design and detailing of the selected energy dissipator.

These activities shall be performed based on the principles and procedures discussed in the following sections.

6. DESIGN PHILOSOPHY

Energy dissipators shall be designed to ensure adequate energy dissipation in a safe, effective and economical manner.

6.1 Safe and Effective Operation

Energy dissipators shall be designed to operate safely and effectively through a wide range of discharges for extended periods of time without having to be shut down for emergency repairs. This requirement shall particularly be met at high discharges to prevent emergency shutdown of overflow sections which, under large floods, could cause overtopping of the river diversion structure and/or unacceptable upstream flooding.

6.2 Degree of Dissipation

The degree of energy dissipation provided downstream of an overflow section shall depend upon the anticipated frequency and duration of operation of these section. For overflow sections that are likely to operate infrequently and/or for short durations, e.g. at plants with a high installed capacity or at headworks which divert most of the river flow for power generation, energy dissipators may be designed to balance their high initial cost against the low net present worth of their repairs. In such instances, tolerating occasional repairs of the dissipators may be more economical than over-investing in an expensive structure designed to withstand the highest flood without damage. However, the structural integrity of the diversion structure as a result to this consideration shall not be compromised.

6.3 Design Discharge

The design discharge for an energy dissipator shall be uniquely determined for each project and shall be dependent upon the damage consequences if this discharge is exceeded. As a general rule, an energy dissipator shall be designed to operate at maximum efficiency in a damage-free condition with discharges at least equal to the magnitude of the inflow design flood for the overflow section. However, it may be designed for a lower flow if operation with the inflow design flood does not create conditions endangering the river diversion structure or causing unacceptable economic damages.

7. TYPES OF ENERGY DISSIPATORS

Energy dissipation below the overflow sections of river diversion structures of run-of-river hydropower projects shall be achieved through one of the following structures:

a. Hydraulic jump type stilling basins.

b. Roller buckets.

c. Deflector or flip buckets.

7.1 Hydraulic Jump Type Stilling Basins

Hydraulic jump type stilling basins shall be used to dissipate the energy of discharge passing the overflow section before the discharge is returned to the downstream river channel. This shall be attained through formation of a controlled hydraulic jump within the confines of the basin over the entire range of flow conditions under which the basin is expected to operate. The design of these basins shall depend on the type of hydraulic jump expected and, hence, on the Froude number of the incoming flow, F_1, computed as

Eq. 1

$$F_r = \frac{v}{\sqrt{gD}}$$

where v is the velocity of flow of the hydraulic jump, D is its depth measured perpendicular to the stilling basin floor and g is the acceleration due to gravity.

7.1.1 Types of Stilling Basins

The hydraulic jump type stilling basin may be provided in one of the following forms:

a. Horizontal apron stilling basin.

b. Sloping apron stilling basin.

The stilling basins shall consist of an apron provided with one or more of the following appurtenances to make them more economical:

a. Chute blocks.

b. Baffle blocks.

c. End sills.

Baffles may be provided on the stilling basin aprons to aid in the formation of the hydraulic jump, reduce its length, decrease the required sequent depth for a given discharge condition and provide stability to the jump. An end sill shall be used to deflect the higher velocity filaments which exist near the basin apron away from the channel bed. It shall act as the terminal wall of the stilling basin and form a step or rise to the channel bed elevation.

7.2 Roller Buckets

Roller buckets shall be used below the overflow sections to dissipate energy in conditions where the tailwater depth is much in excess of the sequent depth. Energy dissipation shall be achieved through two complementary elliptical rollers – a surface roller formed over the bucket and a ground roller formed downstream of the bucket.

7.2.1 Types of Roller Buckets

Roller buckets for energy dissipation may be provided in either of the following forms:

a. Solid roller bucket.

b. Slotted roller bucket.

Both roller buckets shall have upturned buckets for the formation of the rollers. The slotted roller bucket shall have teeth in its bucket to laterally spread the jet passing through it.

7.3 Deflector Buckets

The deflector bucket shall be used to dissipate energy in situations where the tailwater depth is insufficient for the formation of the hydraulic jump. It shall consist of an upturned solid bucket that throws away the incoming flow to a considerable distance downstream of the overflow section as a free discharging upturned jet which falls into the river channel directly, thus avoiding excessive scour immediately downstream of the overflow section. If required, a plunge pool with side and bottom riprap or concrete lining may be excavated in the river bed to prevent severe scour at the point of jet impingement.

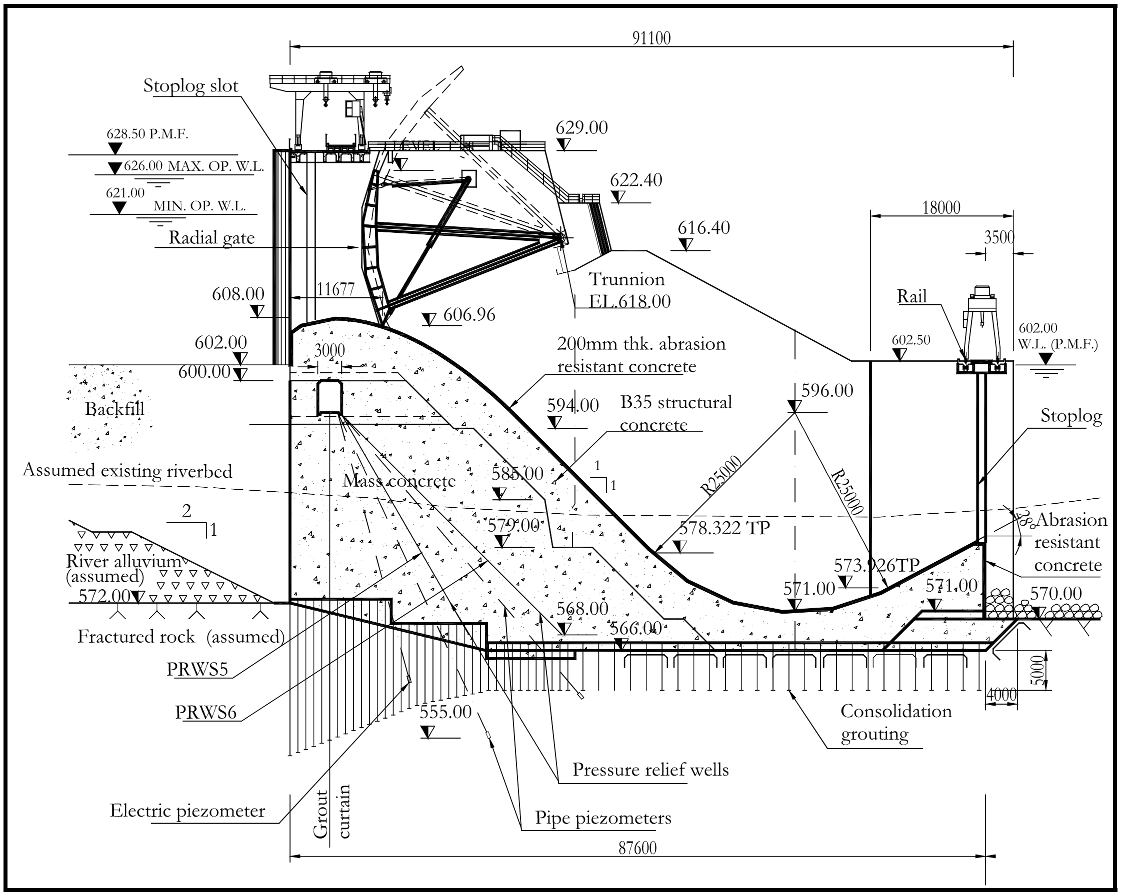


Figure 1: Solid roller bucket at Middle Marsyangdi Hydroelectric Project, Nepal. (courtesy: NEA)

8. SELECTION OF TYPE OF ENERGY DISSIPATOR

Among the various options listed in Section 7, the energy dissipators suitable to a particular site shall be selected considering the following factors:

a. Nature of foundation.

b. Velocity of flow.

c. Tailwater conditions.

d. Topography.
e. Sediment and debris content.
f. Construction considerations.
g. Operation and maintenance.
h. Environmental effects.

Broad guidelines for selection of a particular type of energy dissipator are provided in the following sections. The selected energy dissipator shall be verified through model studies and adopted only after satisfactory results are obtained.

8.1 Nature of Foundation

At sites where the river bed consists of soft, jointed and fractured rock or alluvial deposits, a hydraulic jump type stilling basin, with baffle blocks and an end sill, shall be used even if this requires a relatively longer apron. For small overflow sections where downstream erosion is not potentially dangerous, roller buckets may also be used even if the bed material is loose and erodible.

If the river bed is formed of sound rock, a roller or deflector bucket may be preferred over stilling basins; however, the choice between the buckets and stilling basin shall be based on economic considerations. Deflector buckets may be chosen at sites where erosion-resistant bedrock, capable of absorbing energy by impact, turbulence and friction, exists at shallow depths in the river channel and along its abutments. This condition shall be particularly met for high level deflector buckets to ensure that the scour caused by the impact of the jet on the channel bed does not endanger the diversion structure and other downstream structures, including the deflector buckets, or trigger sliding of the abutment slopes.

8.2 Velocity of Flow

Hydraulic jump type stilling basins, with or without baffles and an end sill, shall generally be preferred for low or intermediate head overflow sections, such as weirs and barrages, where the velocity of the flow is not very high. For such flow velocities, roller buckets may also be considered as an alternative.

For high head overflow sections that witness high velocities, use of baffles and dentations shall be avoided in stilling basins because of their susceptibility to cavitation under high velocity flows. As the absence of baffles and dentations are likely to result in unduly long aprons and high sidewalls, hydraulic jump type stilling basins shall be used with high head overflow sections only if other technically feasible options are not available. Such options include roller buckets or deflector buckets. Of these, deflector buckets shall be preferred at sites where the entering flow is at high velocity and low unit discharge as these conditions result in considerable fraying of the jet by air resistance.

8.3 Tailwater Conditions

For a particular site, the type of energy dissipator and its arrangement shall be decided based on the relationship between the height of the hydraulic jump vis-à-vis the tailwater depth. For this purpose, the following four cases shall be considered:

Case I: Jump height equal, or nearly equal, to tailwater depth

Where the tailwater rating curve approximately follows the hydraulic jump curve or is only slightly above or below it for all discharges, the horizontal apron stilling basin, with or without baffle blocks and end sills, shall be used for energy dissipation.

Case II: Jump height always above tailwater depth

As the tailwater depth shall be insufficient for the formation of a hydraulic jump at the toe of the diversion structure, energy dissipation in this case shall be achieved in one of the following ways:

a. Lowering the apron level of the stilling basin to make the tailwater depth in it equal to the jump height for all discharges, leading to the following alternatives:
    i. Horizontal apron stilling basin with apron depressed below the river bed level.
    ii. Sloping apron stilling basin with depressed floor rising up towards the downstream end.
    iii. Sloping apron stilling basin with depressed floor sloping away from the toe of the overflow section.
b. Horizontal apron stilling basin with baffles or sills at river bed level.
c. Horizontal apron stilling basin with a low subsidiary dam downstream.
d. Deflector bucket.

Case III: Jump height less than tailwater depth

As an imperfect jump would only form under high velocities, one of the following energy dissipators may be used in this case:

a. Sloping apron stilling basin.
b. Roller buckets.

Roller buckets shall be preferred if the tailwater depth is 10 to 20% greater than the sequent depth.

Case IV: Jump height more than tailwater depth at low discharges and less at higher discharges

In this case, enough tailwater depth shall be artificially created to form a jump on the apron at low discharges. For this purpose, one of the following alternatives may be used:

a. Horizontal apron stilling basin with a low secondary dam.
b. Stilling basin with baffle piers or some form of dentated sill.

Case V: Jump height below tailwater depth at low discharges and above at higher discharges

Under these conditions, sufficient tailwater depth may be created to form a jump at high flows using the one of the following methods:

a. Horizontal apron stilling basin with a low secondary dam.
b. Sloping apron stilling basin.

Alternatively, a bucket type energy dissipator that acts as a roller bucket for low discharges and as a deflector bucket at higher discharges may be used provided sound rock conditions exist in the river channel.

8.4 Topography

Where alternatives to the horizontal apron stilling basin are sought, the sloping apron stilling basin shall be preferred over other types of energy dissipators if the natural topography of the site provides a suitable pool and a sloping apron into the pool. For deflector buckets, a sufficiently straight downstream reach of the river shall be preferred.

8.5 Sediment and Debris Content

In view of the high sediment content of Nepali rivers during the monsoon, protrusions in energy dissipators (baffle blocks and dentations in stilling basins and teeth in roller buckets) shall be avoided as they are susceptible to abrasion, and subsequent cavitation, from the high velocity impact of the sediments. In the boulder stages of rivers, these protrusions shall be avoided because large boulders rolling over the overflow section can damage them, rendering them ineffective and increasing their chances of damage by impact, cavitation and abrasion. These protrusions shall also be used with caution in rivers that carry large amounts of trash and floating debris during the monsoon as the trash and debris, particularly logs, may adversely affect the functioning of the basin by blocking the spaces between the protrusions and may even cause structural damage to them. In these conditions, stilling basins having longer aprons, solid roller buckets or deflector buckets shall be preferable.

8.6 Construction Considerations

If the construction area between the diversion structure and the downstream cofferdam is tight, the use of stilling basins may complicate access to the foot of the diversion structure. In addition, as the downstream cofferdam would normally be located below its end sill, a stilling basin may result in longer river diversions than would otherwise be required for the construction of the diversion structure. At such sites, the possibility of using alternatives to stilling basins which do not suffer from these drawbacks shall be explored.

8.7 Operation and Maintenance

From the operation and maintenance perspective, stilling basins without baffle blocks shall generally be given first consideration because of their reliable hydraulic performance and lower maintenance cost. Alternatives shall be resorted to only if the other factors discussed in this section justify their need.

8.8 Environmental Effects

Energy dissipators shall be selected with due thought to their impacts on the environment. For instance, deflector buckets may not be used at sites, especially in cold regions, where the strong air turbulence, local wind currents and spray generated by them are likely to affect the natural environment and manmade structures downstream. Likewise, dissipators that may result in excessive downstream scour and retrogression, thereby damaging nearby structures, shall also be avoided.

9. DESIGN OF HORIZONTAL APRON STILLING BASIN

The design of the horizontal apron stilling basin shall entail hydraulic and structural design of its components, including the basin apron and its appurtenances.

9.1 Hydraulic Design

Hydraulic design of horizontal apron stilling basins shall include determination of the basin floor elevation, length and depth.

9.1.1 Basin I

Basin I shall be used for providing stilling action to the incoming flow. It shall consist of a horizontal apron with an end sill (Figure 2). In boulder reaches of rivers, the sloping face of the end sill shall be kept on the upstream side.

9.1.2 Design Considerations

While selecting the type of stilling basin and its components, the following factors shall be taken into consideration:

a. For F_1<1.7 when proper jump is not formed, bed protection against scour may suffice in many cases. The scour protection shall start from the point of supercritical flow and extend to a distance of about five times the depth of tailwater. However, the protection may vary in type and extent according to the river bed material.

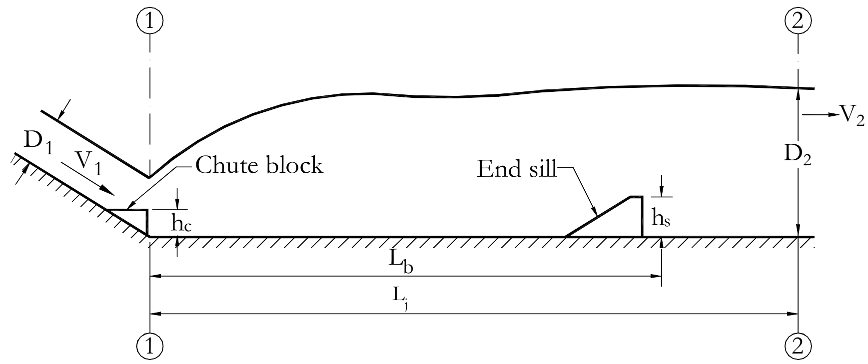


Figure 2: Definition sketch of Basin I

b. As far as possible, stilling basins with F_1 in the range of 2.5 to 4.5 shall be avoided due to the undular nature of the resulting jump. Provided that topography and construction considerations permit, this range may be avoided by modification of the channel width. Where this is not possible, the stilling basin shall be designed with bank erosion protection against wave action downstream of such basin.

c. Ideally, a stilling basin shall be designed for operation in the range 4.5< F_1<9 as the jump is stable and predictable in this range.

d. Flows in the range F_1>9 shall be avoided as they result in a choppy jump which requires unduly long aprons and high sidewalls. If the basin is expected to operate frequently above this limit, it shall be provided with erosion protection up to some distance downstream of the basin. A bucket type dissipator may give comparable results at less cost in such conditions.

9.1.3 Basin Cross Section

For good hydraulic performance, stilling basins shall be provided with a rectangular cross-section. Trapezoidal sections shall not be used as the side eddies created on the surface hinder the formation of effective jumps.

The width of the stilling basin shall be equal to that of the discharge carrier. Expansions or contractions in the design width of supercritical flow shall be avoided. However, the expansion towards end sill shall be hydraulically effective and efficient.

9.1.4 Apron Elevation

The apron elevation of stilling basins shall be calculated by either deducting the specific energy at section 1-1 (Figure 2) from the total energy line at that section or the specific energy at section 2-2 from the downstream total energy line. For this purpose, the sequent depths of the hydraulic jump shall be computed from Eq. 2 and Eq. 3 (IS: 4997 – 1968).

Eq. 2
$$H_L = \frac{(D_2 - D_1)^3}{4D_1D_2}$$

Eq. 3
$$D_2 = \frac{1}{2}D_1\left[\sqrt{1+\frac{8q^2}{gD_1^3}} - 1\right]$$

Eq. 4
$$D_1 \approx D_c$$

where H_L is the head loss in the hydraulic jump, D_2 is the depth sequent to D_1 for the horizontal apron, D_c is the critical water depth and q is the discharge intensity of the incoming flow.

The head loss, H_L, shall generally be assumed equal to the difference in the upstream and downstream total energy lines. If it is considered that friction forces will reduce the velocity of the incoming flow, a more accurate value of H_L shall be computed iteratively by first estimating D_1 from the assumed H_L, estimating friction losses on the glacis using equations of gradually varied flow and computing a more accurate value of H_L and consequently that of D_1 till the desired accuracy is achieved.

9.1.4.1 Basin Length and Depth

The length of the basin shall be determined from the curve given in Figure 3. Generally, the basin floor shall not be raised above the level required from sequent depth consideration. If site conditions require the floor to be raised above this level, the raise shall not exceed 15 percent of D_2, and the basin shall be supplemented with chute blocks and baffle blocks. However, if the velocity of flow exceeds 15 m/s at their location or water carrying heavy silt, baffle blocks shall not be used, and the floor of the basin shall be kept at a depth equal to D_2 below the tailwater level. The tail-water depth shall not generally exceed 10 percent of D_2.

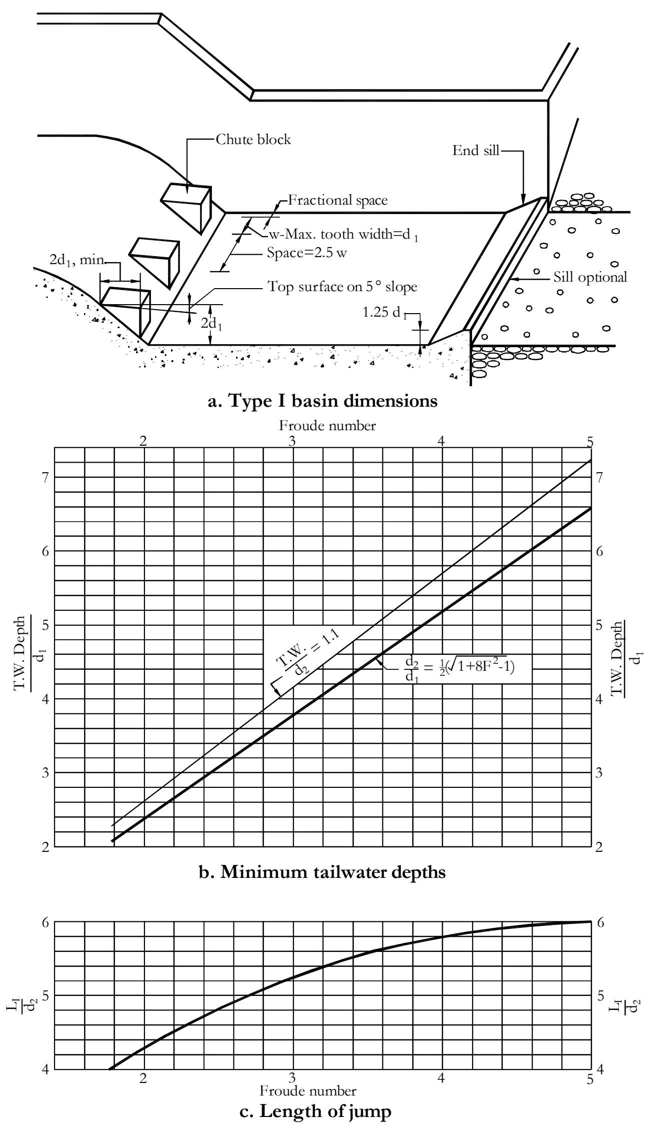


Figure 3: Basin I type stilling basin (USBR, 1978)

9.1.5 Side Walls

To maintain a rectangular cross-section, the stilling basin shall be provided with vertical side walls. However, if necessary for construction purposes, a small slope, around 1:8, from the vertical may be provided to these walls. If the slopes exceed this value, the basin shall be subject to model testing to check for cavitation damage and streamline disturbance along the side walls. When sloped walls are required, the width of the basin at its mid-height shall be equal to the width of the overflow section so as to minimize expansion and contraction of flow at the design discharge.

9.1.5.1 Freeboard

The side walls of the stilling basin shall have sufficient freeboard so that they are not overtopped by surges, splashes, sprays and wave action set up by the turbulence of the jump. The free board should be not less than the velocity head:

Eq. 5

$$H_f = \frac{v^2}{2g}$$

where H_f is the freeboard in m, v is the flow velocity in the basin in m/s and g is the acceleration due to gravity.

9.1.6 Drainage

Adequate drainage shall be provided under the stilling basin apron to reduce uplift pressures below it. Surface drainage shall be provided to facilitate inspection and maintenance. Both types of drainage shall be designed in accordance with the provisions of Section 13.

9.1.7 Model Studies

As a general rule, physical model testing of stilling basins shall be performed, preferably with mobile body modeling to study potential erosion of material downstream and to the sides of the basin. The tests may be dropped only if the data on tailwater levels and other operating conditions are reliable. However, tests shall be mandatory to confirm the design of the basins if anyone of the following conditions exists:

a. The jet entering the basin is non-uniform in regard to both velocity and depth.
b. The fall of the flow exceeds 15 m.
c. Discharge intensities are greater than 30 $m^3/s/m$.
d. The flow is likely to be asymmetric.

9.2 Structural Design

Structural design of the horizontal apron stilling basin shall include design of the basin floor slab and its appurtenances. The design shall be performed in accordance with considerations discussed in the following sections.

9.2.1 Basin Floor Slab

Structural design of the basin floor slab shall consist of determining the floor slab thickness, its reinforcement and its anchorage to the underlying rock.

9.2.1.1 Design Loads and Load Cases

The stilling basin floor slab shall be designed for the following forces:

a. Hydrostatic uplift.
b. Pounding and vibrations from hydrodynamic forces.
c. Forces generated from differential settlement.

The floor slab shall be designed for the load cases listed in Table 1. These load cases assume the subsurface drains of the basin to be effective.

Table 1: Load cases for design of stilling basin floor slab

Case	Description	Loads
I	Stilling basin operating at inflow design flood (Figure 4).	• Dead load of floor slab. • Hydrostatic load due to water column above basin, varying between D_1 and D_2 corresponding to design discharge. • Uplift force equal to ½ of tailwater depth over slab base.
II	Reservoir at full reservoir level with gates closed and stilling basin empty (Figure 5).	• Dead load of floor slab. • Uplift force equal to ½ of tailwater depth over slab base.

(Based on IS: 11527 – 1985)

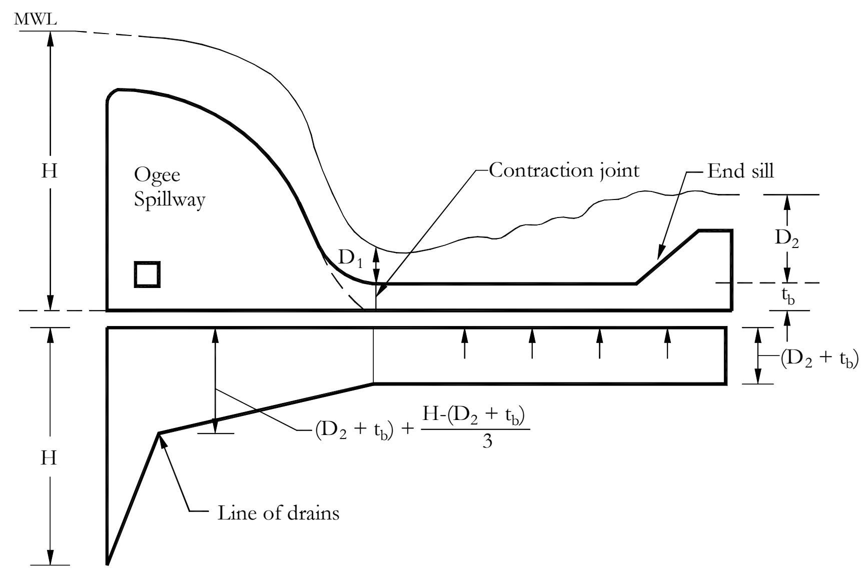


Figure 4: Uplift forces for Case I (IS: 11527 – 1985)

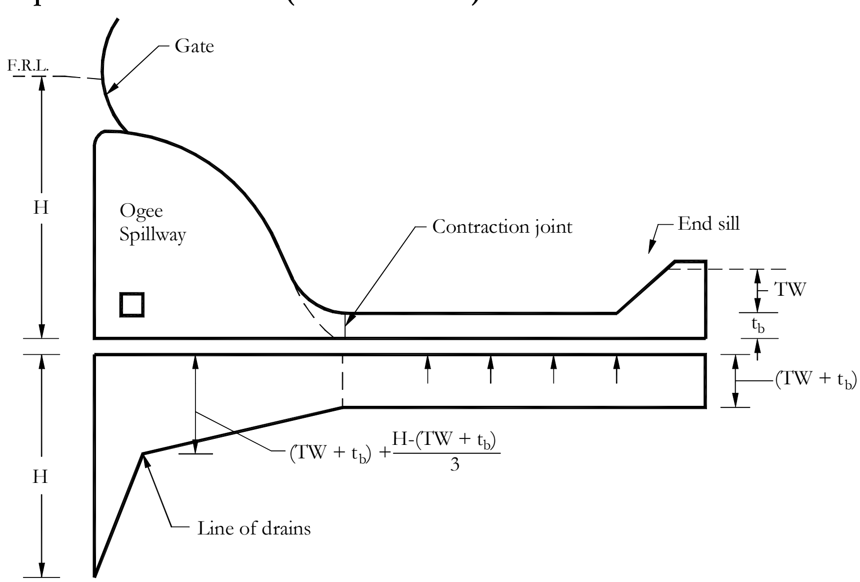


Figure 5: Uplift forces for Case II (IS: 11527 – 1985)

9.2.1.2 Basin Floor Slab Thickness

The thickness of floor slab shall be determined considering the load cases listed in Table 1 and the differential foundation movement, if any. In any case, the slab thickness shall not be less than 1000 mm.

9.2.1.3 Floor Slab Arrangement

To avoid shrinkage and temperature cracks, the basin slab shall be divided into independent, approximately square panels by contraction joints parallel and perpendicular to the channel or the basin centre line. The size of these panels shall be adequate to resist the distorting hydrodynamic forces. The sizing of the panel shall also take into consideration the ability of the available construction equipment to complete concreting of the entire panel in a single stretch. The panels shall be cast in alternate bays with construction joints.

On relatively yielding rock foundations where differential movement of adjacent panels is a possibility, a key shall be formed along each transverse contraction joint.

9.2.1.4 Floor Slab Reinforcement

In thick basin slabs resting on rock foundations normally covered with tail water, structural reinforcement for the slab may not be necessary. The uplift on the slab shall be resisted by adequate anchoring to the underlying rock.

The independent slab panels shall be reinforced with minimum steel to prevent cracks due to shrinkage and temperature stresses not relieved by contraction joints and to avoid cracks from differential settlement on yielding foundations. The minimum reinforcement for such panels shall be 20 mm diameter bars at 300 mm centers in both directions. Additional reinforcement shall be provided for slab panels on unfavorable foundation conditions or subject to high hydrostatic uplift pressure.

Reinforcing steel provided in the panels shall not extend across the contraction joints. On unyielding foundations, the panels may be reinforced on the top face only if the bond between the concrete and rock at the bottom face appear adequate to distribute the shrinkage cracks and to minimize bending stresses in the anchored slab for the assumed uplift head.

9.2.1.5 Grade of Concrete and Reinforcement Cover

M30 grade concrete may be used for the basin slabs if the drop between the crest and the apron level is up to 40 m. For drops greater than 40 m, M40 grade concrete shall be used. The reinforcement shall have a minimum cover of 100 mm.

9.2.1.6 Floor Slab Anchorage

Anchors for the floor slab shall normally be designed for Case I and checked for Case II (Table 1). The number of anchors required to tie the slab to the rock shall be determined for the net uplift force applying a factor of safety of 1.2 on the tensile yield strength of the reinforcing steel. Similarly, the length of the anchors shall be found based on the following criteria:

a. The bond stress between the steel and grout as well as the grout and rock generated by the actual force in each anchor shall be less than the corresponding permissible bond stress.
b. The rock mass into which the anchor is grouted shall not be dislodged by the uplift pressure.

For checking the first criteria, the values of permissible bond stress shall be based on actual tests; however, in the absence of tests data, a grout proportion of 1:2 with permissible bond stress between steel and grout equal to 0.6 MPa and between grout and rock equal to 0.4

MPa may be assumed. To meet the second criteria, the anchors shall be sufficiently long to engage a conical mass of rock, with a vertex angle of 45º, whose submerged weight is able to withstand the net upward force (Figure 6).

The minimum length of the anchors shall be 3 m, and their diameter shall not be less than 25 mm. The diameter of grout hole shall be not less than 1.5 times diameter of the anchor bar designed. The anchor bars shall be staggered in plan, and their spacing shall not exceed 3 m centers.

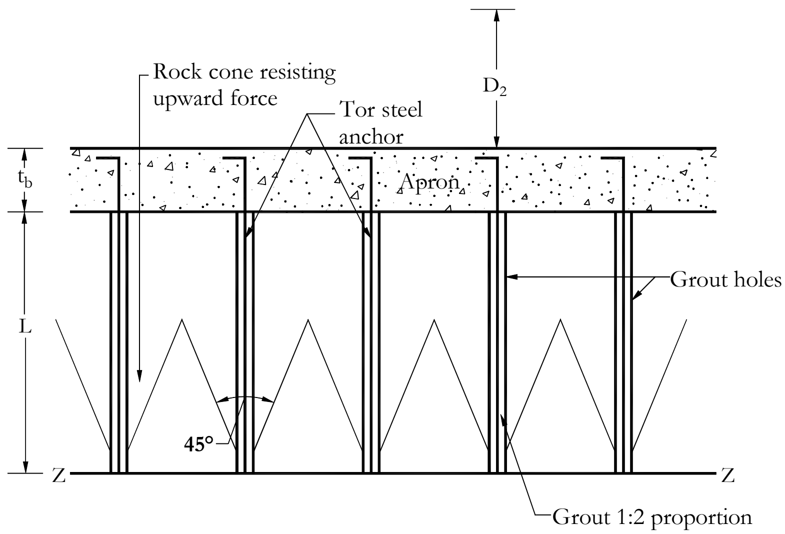


Figure 6: Details of anchors and grout holes

10. DESIGN OF SLOPING APRON STILLING BASIN

The design of a sloping apron stilling basin shall comprise hydraulic and structural design of its apron.

10.1 Hydraulic Design

Hydraulic design of sloping apron stilling basins shall include selection of the type of stilling basin and determination of its floor slope, overall shape and length. The slope and overall shape of the apron shall be determined from economic considerations while its length shall be fixed based on the type and soundness of the downstream river bed.

10.1.1 Selection of Stilling Basin

Two types of stilling basins, viz. Basin II (Figure 7) and Basin III (Figure 8), may be used for energy dissipation on sloping aprons. As indicated below, the choice between the two types shall be based on relation between the sequent depth D_2 and the tailwater depth indicated in Table 2:

Table 2: Criteria for selection of basin type

Basin type	Condition of application
Basin II	Tailwater depth higher than the D_2 curve at all discharges.
Basin III	Tailwater depth at maximum discharge exceeds D_2 considerably but is equal to or slightly greater than D_2 at lower discharges

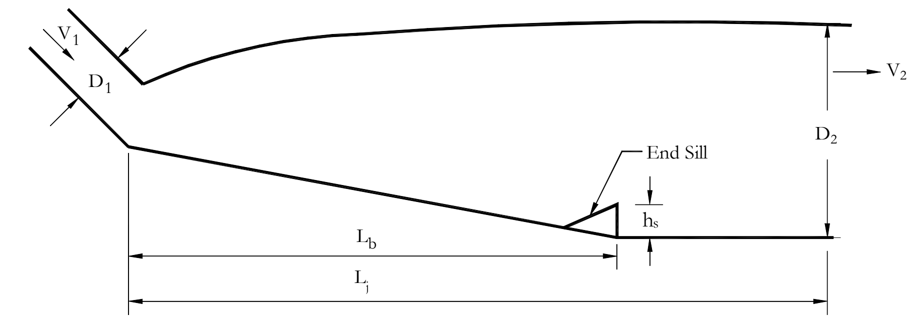


Figure 7: Definition sketch of Basin II

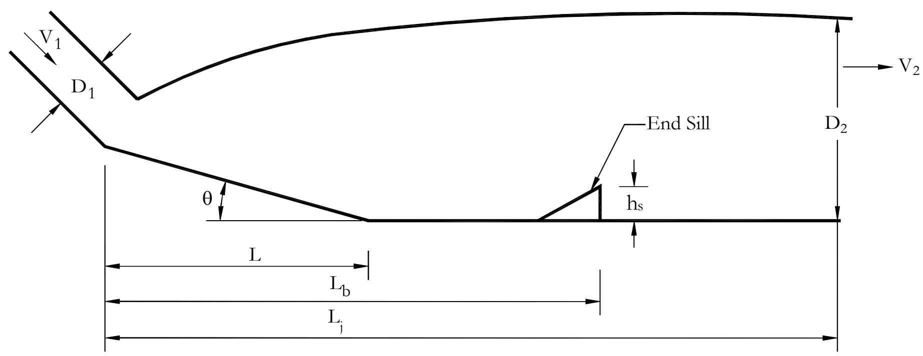


Figure 8: Definition sketch of Basin III

10.1.2 Basin II

Basin II shall consist of a sloping apron with a solid or dentated end sill (Figure 7).

10.1.2.1 Basin Slope, Shape and Length

The slope, overall shape and length of the basin shall be designed through the following iterative procedure:

a. A level at which the front of jump could form for maximum tailwater depth and discharge shall be assumed.

b. From the known upstream total energy line, D_1 shall be determined through Bernoulli's theorem, and F_1 shall be calculated using Eq. 1. Thereafter, conjugate depth D_1 shall be found from Eq. 6 for horizontal aprons and Eq. 7 for sloping aprons (IS: 4997 – 1968).

Eq. 6
$$\frac{D_2}{D_1} = \frac{1}{2}\left[\sqrt{1+8F_1^2} - 1\right]$$

Eq. 7
$$\frac{D_2^{'}}{D_1} = \frac{1}{2\cos\theta}\left[\left(\frac{8F_1^2\cos^3\theta}{1-2K\tan\theta}+1\right)^{\frac{1}{2}} - 1\right]$$

where $D_2^{'}$ is the depth sequent to D_1 for horizontal apron, θ is the slope angle of basin and K is the shape factor determined from .

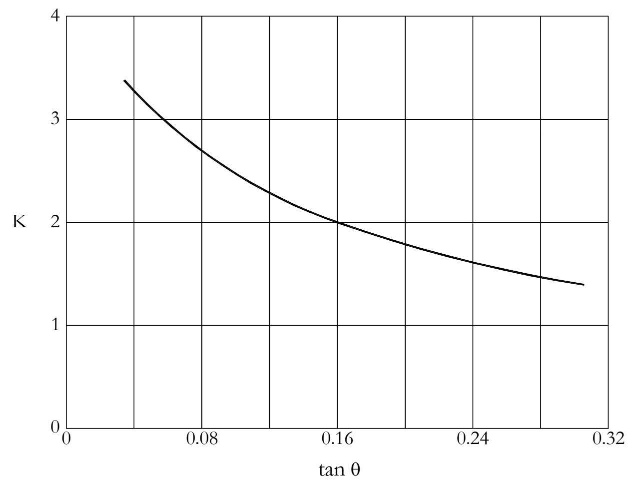


Figure 9: Curve for determination of shape factor (IS: 4997 – 1968)

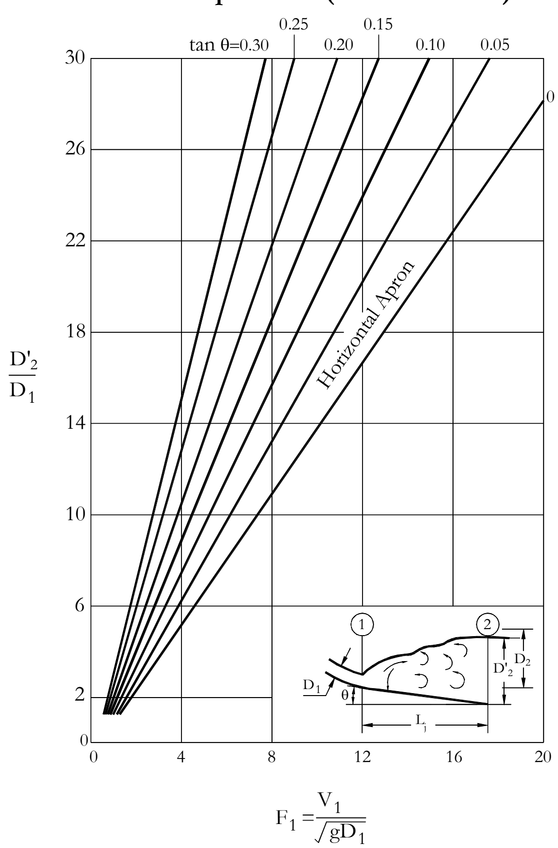


Figure 10: Curve for determination of conjugate depth ratio for Basin II (IS: 4997 – 1968)

c. A certain slope, θ, shall be assumed, and the values of the conjugate depth D'_2 and the length of the jump, L_j, shall be determined for the above F_1 from and , respectively. The length of the apron, L_b, shall be kept equal to 60 percent of L_j.

d. The conjugate depth D'_2 shall be checked against the available tailwater depth at the end of the apron. If the two do not match, the slope or the level of the upstream end of the apron or both shall be changed, and steps a to c shall be repeated till the slope and the location of the apron are compatible with the hydraulic requirements.

e. The apron designed for maximum discharge may then be tested at lower discharges, say at ¼, ½ and ¾ of the maximum discharge. If the tailwater depth is sufficient or in excess of the conjugate depth for the intermediate discharges, the design shall be considered acceptable. If not, either a flatter slope with a lower apron level shall be tried or Basin IV may be adopted.

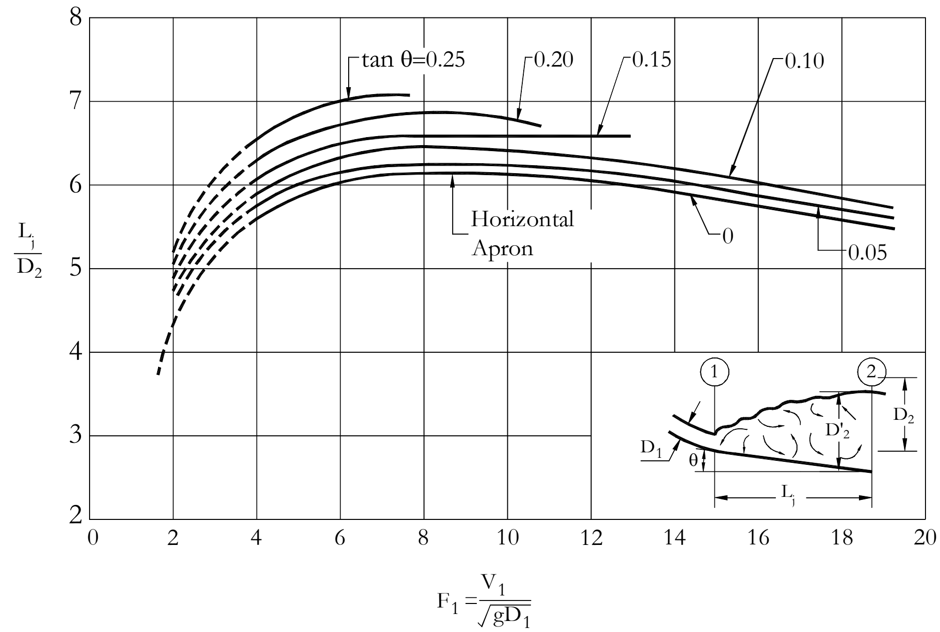


Figure 11: Curve for determining length of hydraulic jump for Basin II (IS: 4997 – 1968)

10.1.2.2 Appurtenances

The solid or dentated end sill shall have a height equal to 0.05 to 0.2 D_2. Its upstream slope shall be of 1:2 (vertical to horizontal) to 1:3.

10.1.3 Basin III

Basin III shall consist of an apron that is partly sloping and partly horizontal. It shall have a a solid or dentated end sill (Figure 8).

10.1.3.1 Basin Slope, Shape and Length

The slope, overall shape and length of Basin III shall be designed through the following iterative procedure:

a. The discharge at which the tailwater depth is most deficient shall be determined.

b. For this discharge, the level and length of the apron shall be determined based on the criteria given in Section 8.8.

c. A level at which the hydraulic jump can form for the maximum tailwater depth and discharge shall be assumed.

d. From the known upstream total energy line, D_1 shall be determined through Bernoulli's theorem, and F_1 shall be calculated using Eq. 1. Thereafter, conjugate depth D_2 shall be found from Eq. 6 for horizontal aprons and Eq. 7 for sloping aprons.

e. Through trial and error, a suitable slope, θ, shall be determined so that the available tailwater depth matches the required conjugate depth D'_2 determined from Figure 12.

f. The length of the jump, L_j, for the above slope shall be determined from . If the sum of the lengths of the inclined and horizontal portions equals about 60 percent of L_j, the design shall be considered acceptable. If not, fresh trials may be carried out by changing the level of the upstream end of the jump formation.

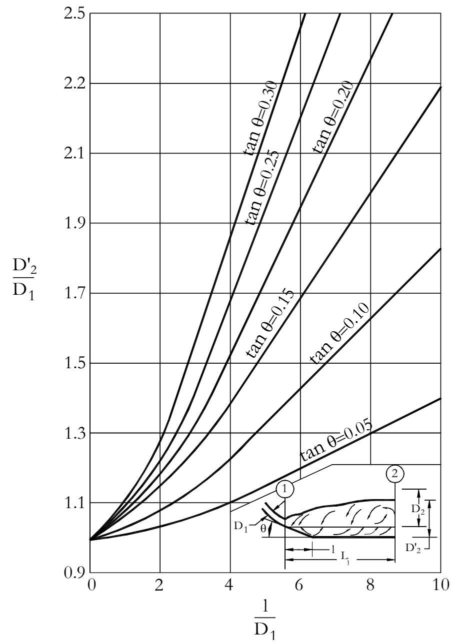


Figure 12: Tailwater requirement for Basin III (IS: 4997 – 1968)

10.1.3.2 Appurtenances

The solid end sill shall have a height equal to 0.05 to 0.2 D_2. Its upstream slope shall be of 1:2 (vertical to horizontal) to 1:3.

10.2 Structural Design

The structural design of sloping apron stilling basin shall follow the general design principles for the horizontal apron stilling basins discussed in Section 9.2.

11. DESIGN OF SOLID ROLLER BUCKETS

The design of a solid roller bucket shall include hydraulic and structural design of the roller bucket.

11.1 Hydraulic Design

The design of a solid roller bucket shall be aimed at forming both the surface roller and the ground roller properly so that effective energy dissipation can be achieved. The design shall ensure that neither the surface roller nor the ground roller is inhibited so that the hydraulic phenomenon of sweep out or heavy submergence does not occur.

Hydraulic design of the solid roller bucket shall consist of determination of invert elevation, radius and lip angle of the bucket. It shall also include fixation of the downstream bed level and its protection.

11.1.1 Bucket Invert Elevation

The bucket invert elevation shall be fixed by assuming an invert level and checking whether the assumed level satisfies the following conditions (IS: 7365 – 1985):

a. $D_3/D_2 = 1.1 – 1.4$

b. $h_r/D_3 = 0.75 – 0.9$

where D_3 is the difference in the design maximum tailwater level and the bucket invert level, h_r is the roller height measured from the invert and D_2 is the sequent depth (). If both of these conditions are satisfied, the surge height, h_s, measured above the invert level will be 105 to 130 percent of D_3, i.e. $h_s / D_3 = 1.05$ to 1.3.

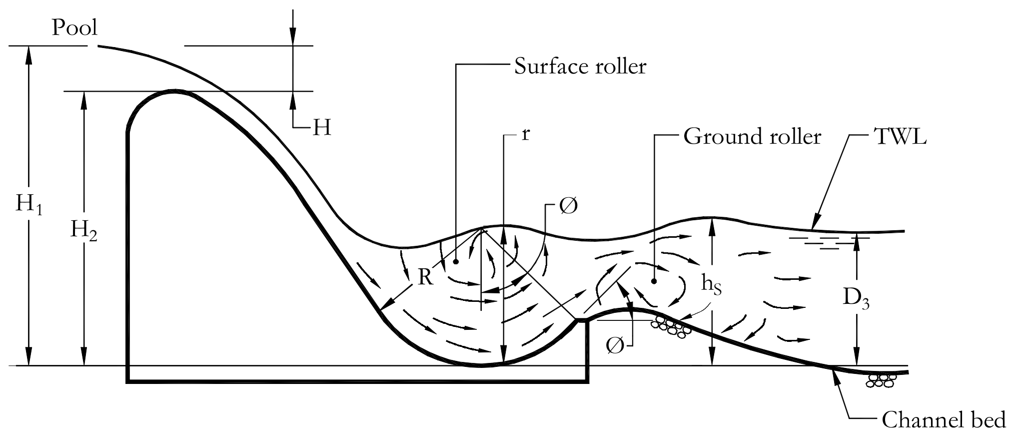


Figure 13: Definition sketch of solid roller bucket

The parameters D_2 and h_r required to check the above conditions shall be computed using the design charts given in Figure 14 and Figure 15, respectively. For the known maximum reservoir water level, spillway crest elevation, tailwater depth and specific discharge, D_2 shall be estimated from Eq. 6 with F_1 computed from Eq. 1 and D_1 from the relation

Eq. 8 $$D_1 = \frac{q}{v_a}$$

where v_a, the actual velocity of the incoming flow, shall be found from Figure 14 with v_t, the theoretical velocity of the incoming flow, calculated as

Eq. 9 $$v_t = \sqrt{2gH_3}$$

in which H_3 is the difference in the reservoir pool elevation and the tailwater depth.

Similarly, h_r shall be estimated from Figure 15 for the appropriate discharge parameter, F_D, computed using the relation

Eq. 10 $$F_D = \frac{q}{\sqrt{gH_1^{3/2}}} \times 10^3$$

where H_1 is the difference in pool elevation and bucket invert elevation.

If the need to attain h_r/D_3 between 0.75 and 0.90 causes the bucket invert elevation to be considerably lower than the channel bed, substantial excavation may be required to lower the channel bed for minimizing the entry of downstream bed material into the bucket by the ground roller (Section 11.1.8). In such cases, the invert may be brought near to the channel bed level by fixing D_3/D_2 between 1.2 to 1.4 and maintaining h_r/D_3 between the prescribed limits.

While computing the invert level, the full range of overflow discharge shall be investigated. When h_r is less than $0.2D_3$ in the case of falling tailwater and $0.3D_3$ in the case of rising tailwater, free jet condition may be expected (Figure 15).

While using the design charts in Figure 14 and Figure 15, note shall be taken of the fact that the charts are applicable for the ranges of variables indicated therein.

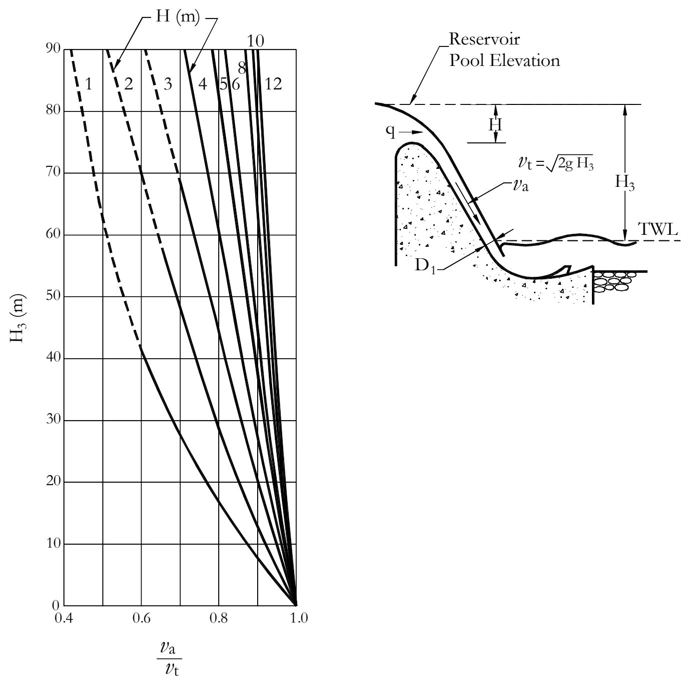


Figure 14: Curve for determination of velocity of jet entering bucket (IS: 7365 – 1985)

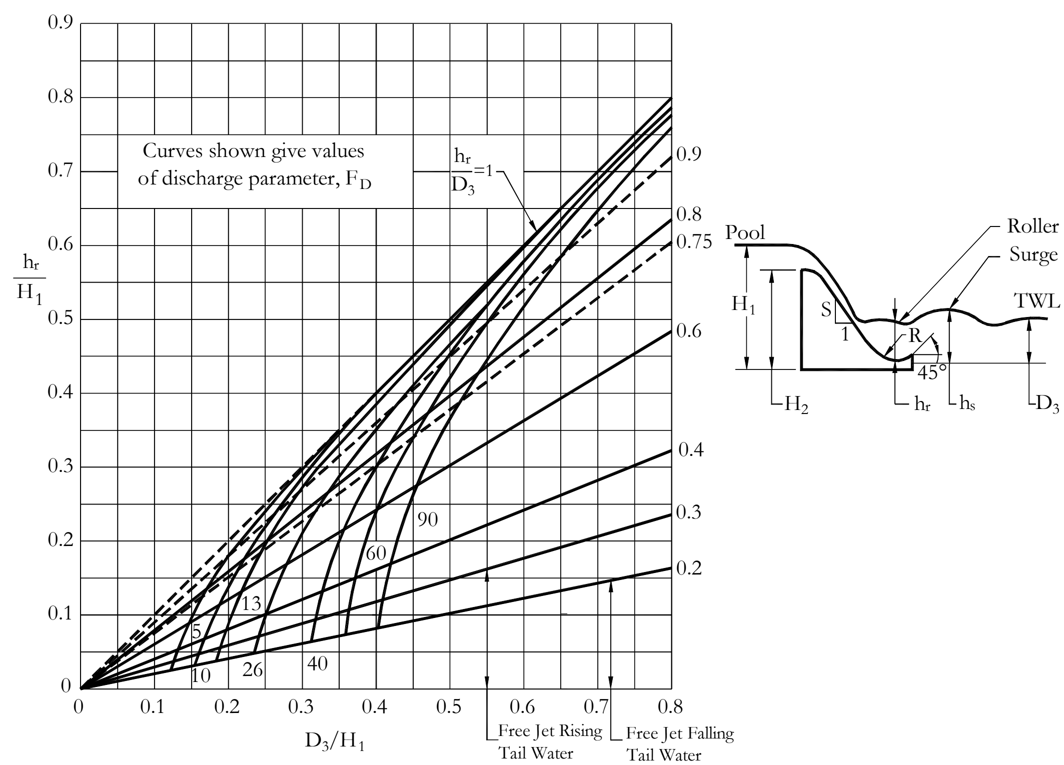


Figure 15: Curve for determination of roller depth for solid roller bucket (IS: 7365 – 1985)

11.1.2 Bucket Radius

The radius of the bucket, *R*, may be computed using any one of the formulae listed below. When the bucket lip angle is 45°, the following formula may be used (IS: 7365 – 1985):

Eq. 11 $$\frac{R}{H_1} = 8.26 \times 10^{-2} + 2.07 \times 10^{-3} F_D + 1.4 \times 10^{-5} F_D^2$$

where F_D shall be obtained from Eq. 10. Alternatively, the value of R, in m, may also be computed from Eq. 12, Eq. 13 or Eq. 14 (IS: 7365 – 1985).

Eq. 12 $$R = 0.305 \times 10^{\left(\frac{v_t + 6.4H + 4.88}{3.6H + 19.5}\right)}$$

Eq. 13 $$\sqrt{F_1} = 0.09 \frac{R}{D_1} + 1.96$$

Eq. 14 $$F_1 = 13.0 R^{\frac{1}{4}} - 19.50$$

In the above equations, H is the depth of flow over the overflow section in m.

The value of *R* thus computed shall satisfy the specified range of H_1/R in Table 3 for which good roller can be expected, consistent with economic and structural considerations. It shall also not be less than the minimum allowable radius.

Table 3: Range of H_1/R for satisfactory bucket performance

Discharge parameter, F_D	Equivalent Froude Number, F_1	Range of H_1/R
90	5.4	2 – 5
60	6.7	2 – 6
40	8.3	2 – 6
26	10.3	3 – 6
13	14.7	3 – 8

(Source: IS: 7365 – 1985)

While using the ranges specified in Table 3, the following factors shall be borne in mind:

a. The maximum values of H_1/R recommended in the table indicate commencement of a pulsating surge downstream of the bucket or commencement of a sloping channel type jump apparently uninfluenced by the presence of the bucket. The minimum values of H_1/R may indicate commencement of eddies in the bucket region and also result in an increase of the bucket cost. The bucket, therefore, shall neither be too deep to set up vortices within the bucket nor be too shallow to allow eddies to form at the junction of the overflow section with the upstream end of the bucket circle.

b. The decrease in radius within the range of good roller action increases scour. The rate of increase, however, diminishes with higher values of Froude number.

11.1.3 Bucket Lip

The design of the bucket lip shall consist of fixing its shape and width, lip angle and lip height.

11.1.3.1 Shape and Width

The top surface of the bucket lip shall have a downstream slope of 1 in 10, or slightly more, to prevent erosion near the lip (). The width of the lip shall not exceed one tenth of the bucket radius, *R*; however, it shall not be less than 1 m.

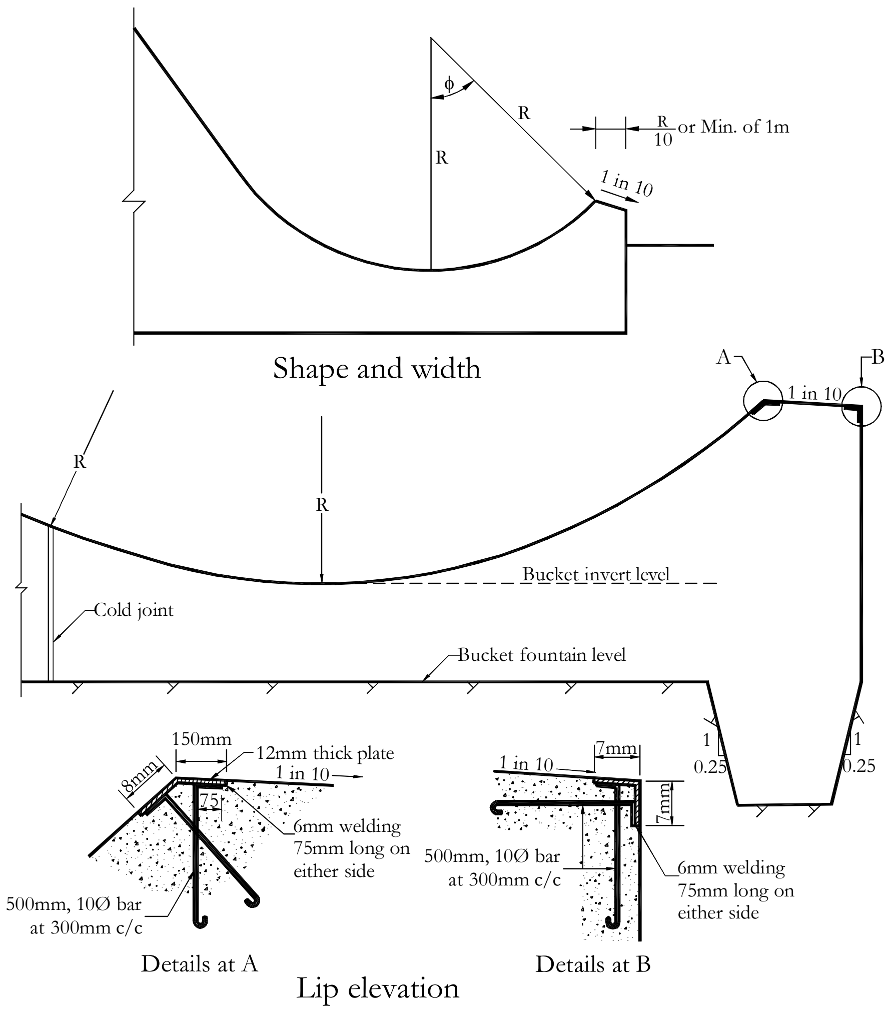


Figure 16: Shape and width of bucket lip (IS: 7365 – 1985)

To avoid downstream debris from riding over it, the downstream face of the lip may be kept vertical in about 1.0 m depth from top of the lip. A key may be provided below the bucket end sill to protect the bucket against the scour, its depth being fixed based on the depth of scour or model studies. Steel plates and angles may be fixed at the edges of the lip for their protection.

11.1.3.2 Lip Angle

A bucket lip angle, ϕ, of 45° with the horizontal shall generally be satisfactory for most cases where F_D lies between 30 and 80. In other cases, a smaller lip angle, up to 35°, may be economical and more desirable as it would need a lesser depth of tailwater for roller action to begin; however, the reduced angle shall be adopted after model tests.

11.1.3.3 Lip Height

Although the height of the lip from the bucket invert level shall depend on the R and ϕ, the lip level shall be kept slightly higher than the bed level downstream to avoid entry of bed material in the bucket. If possible, the height of the bucket lip above the invert level may be kept approximately equal to one-sixth of the maximum tailwater depth. Reference to Section 11.1.8 shall be made in this connection.

11.1.4 Pressures on Bucket

At any point on the solid roller bucket, the pressures shall nearly be equal to the hydrostatic head measured as the difference in tailwater elevation and the elevation of the point.

11.1.5 Training Walls

The design of training walls for the solid roller bucket shall be based on the procedure described in Section training walls. Where required, model tests may be performed to verify the design thus arrived at. If the overflow needs to be separated from an adjoining embankment, canal or other structures, the optimum lengths and top levels of the walls shall be decided based on the water and scour profiles obtained from model tests.

11.1.6 Divide Walls

If adjoining buckets below an overflow section are located at different elevations, divide walls shall be provided to separate the flow from these buckets. The top levels and lengths of the divide walls shall be decided based on model studies.

11.1.7 Drainage

Subsurface drainage shall be provided under the bucket to reduce uplift pressures below it. Likewise, surface drainage shall be needed to prevent impoundment of water in the bucket which could result in formation of ice in cold weather or stagnant smelly pools in hotter climates. It shall also be required to facilitate inspection and maintenance. These drainage systems shall be provided in accordance with the details prescribed in Section 13.

11.1.8 Downstream Bed Level and Protection

To prevent the damage to the bucket from the churning action of downstream bed material brought into it by ground rollers, the channel bed immediately downstream of the bucket shall be set at 1 to 1.5 m below the lip level. Where the invert of the bucket is required to be set below the channel general bed level, the channel shall be dressed down in one level to about 1 to 1.5 m below the lip level in about 15 m length downstream, and a recovery slope of 1 in 3 (vertical to horizontal) shall be provided to meet the general bed level (Figure 17). Careful model studies shall be done to check this tendency. If possible, provision of a solid apron or cement concrete blocks may be considered to avoid trapping of the river bed material in the bucket.

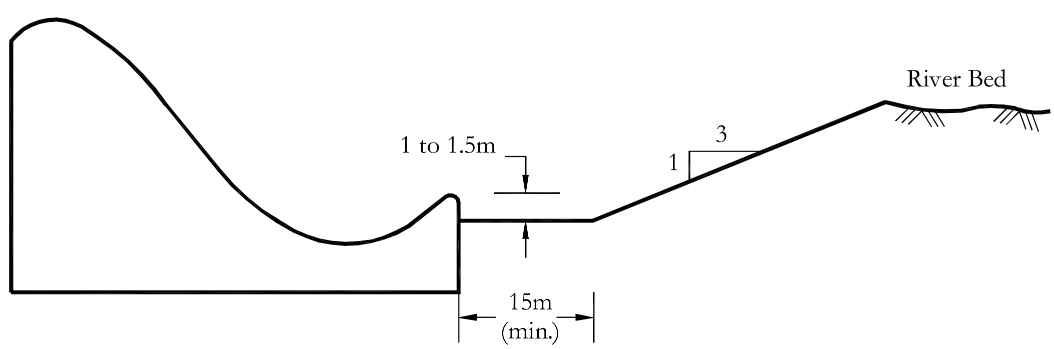


Figure 17: Recovery slope downstream of solid roller bucket

Where the bucket invert is located substantially higher than the general channel bed level and the channel bed is erodible, a concrete apron with a minimum length of 15 m shall be provided downstream of the bucket lip to prevent deep scour very close to the lip caused by cascading flows over it for small discharges (Figure 18). The apron shall be keyed into good rock.

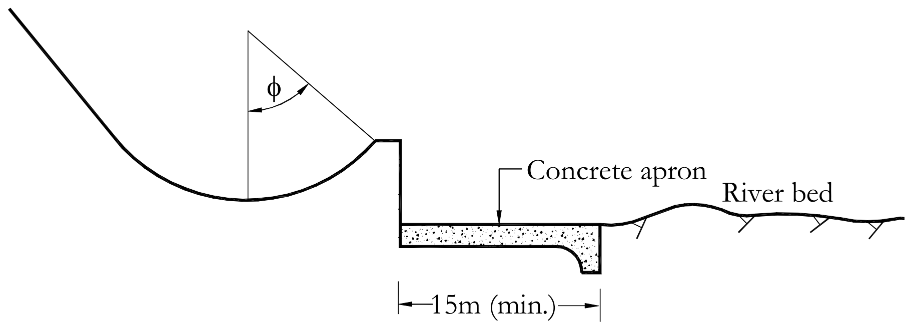


Figure 18: Concrete apron downstream of bucket lip

11.1.9 Model Studies

Model tests on the roller bucket shall be performed to confirm its design if anyone of the following conditions exists:

a. Sustained operation near limiting conditions is expected.

b. Discharge intensity of the bucket exceeds 30 $m^3/s/m$.

c. Velocity of flow entering the bucket exceeds 15 m/s.

d. Potential for eddies downstream of overflow section exists.

e. Waves in the downstream channel are likely to cause problems like unstable flow and flow disturbances.

11.2 Structural Design

Structural design of the solid roller bucket shall include design of the bucket reinforcement and the anchorage of bucket to the underlying strata. The design shall be performed in accordance with considerations discussed in the following sections and in conformity with the requirements.

11.2.1 Design Forces

The roller bucket shall be designed to withstand the horizontal force and moment on the bucket due to the change in momentum of the flowing water. The total horizontal force on the bucket lip, and the corresponding moment about a plane A-A tangential to the bucket invert (), shall be computed from the following equations (IS: 11527 – 1985):

Eq. 15 $$F = \frac{\gamma_w \; b \, v_a}{g}(1 - \cos\phi)$$

Eq. 16 $$M = 0.5FR(1 - \cos\phi)$$

where F is the horizontal force on bucket lip in kN, M is the moment about a plane tangential to the bucket invert in kN-m, γ_w is the unit weight of water in kN/m^3, b is the bucket width in m, v_a is the actual velocity of flow entering bucket in m/s, ϕ is the bucket lip angle, g is the acceleration due to gravity in m/s^2 and R is the radius of bucket in m.

11.2.2 Bucket Reinforcement

The reinforcement required to resist the moment M shall be found using the working stress method. For this, the effective depth of bucket shall be computed using the relation (IS: 11527 – 1985)

Eq. 17 $$d = \sqrt{R^2 + \left(R\sin\phi + t_w\right)^2} - R - d^{'}$$

where d is the effective depth of the bucket in m (), t_w is the width of bucket lip in m and d' is the effective cover to reinforcement in m.

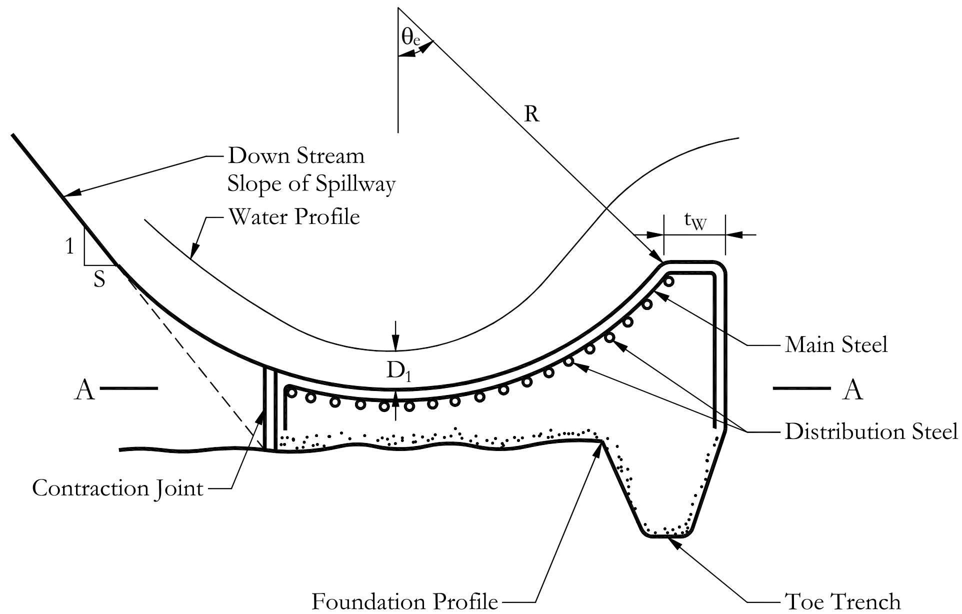


Figure 19: Typical section of solid roller bucket (IS: 11527 – 1985)

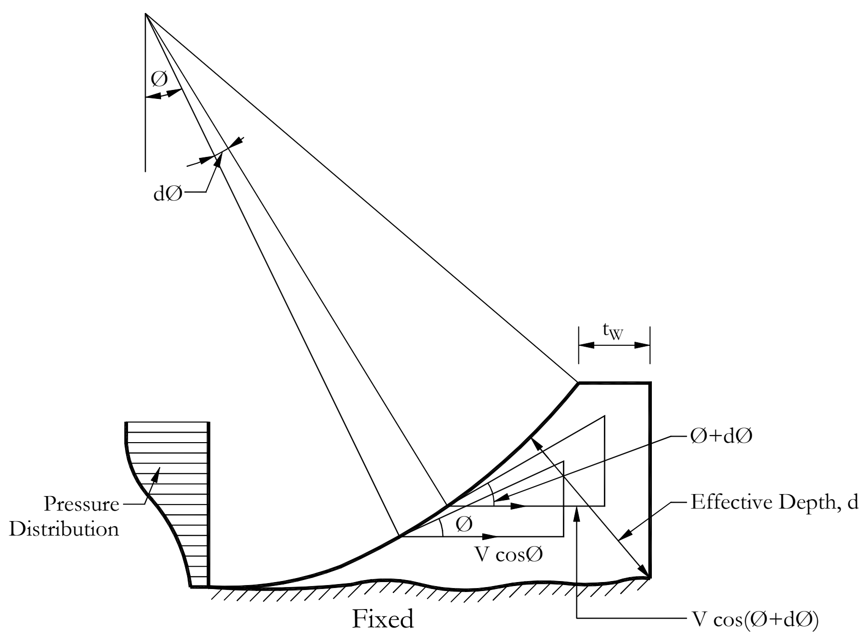


Figure 20: Forces on the bucket due to flowing water (IS: 11527 – 1985)

The bucket shall be provided with a minimum longitudinal reinforcement consisting of 20 mm diameter bars at 300 mm centers. Distribution reinforcement in the transverse direction shall be equal to 20 percent of the longitudinal reinforcement; however, this shall not be less than 16 mm diameter bars at 300 mm centers.

11.2.3 Bucket Anchorage

The bucket anchorage shall be computed for 50 percent of the uplift force assuming the subsurface drainage system provided under the bucket to be fully effective. Accordingly, the anchors shall be designed for the force given by (IS: 11527 – 1985)

Eq. 18
$$F_u = R\left[0.5\,\beta\,\gamma_w - \gamma_c\alpha - \gamma_c\left\{1 - \frac{0.25(\sin 2\phi + \sin 2\varphi) + 0.5(\phi + \varphi)}{(\sin\phi + \sin\varphi)}\right\}\right]$$

where F_u is the uplift force to be resisted by anchors in kN, R is the radius of bucket in m, β is the ratio of the uplift head U_h to bucket radius R (Figure 21), γ_w is the unit weight of water in kN/m^3, γ_c is the unit weight of concrete in kN/m^3, α is the ratio of bucket thickness at invert elevation d_t (Figure 21) to bucket radius R, ϕ is the bucket lip angle and φ is the inlet angle of bucket.

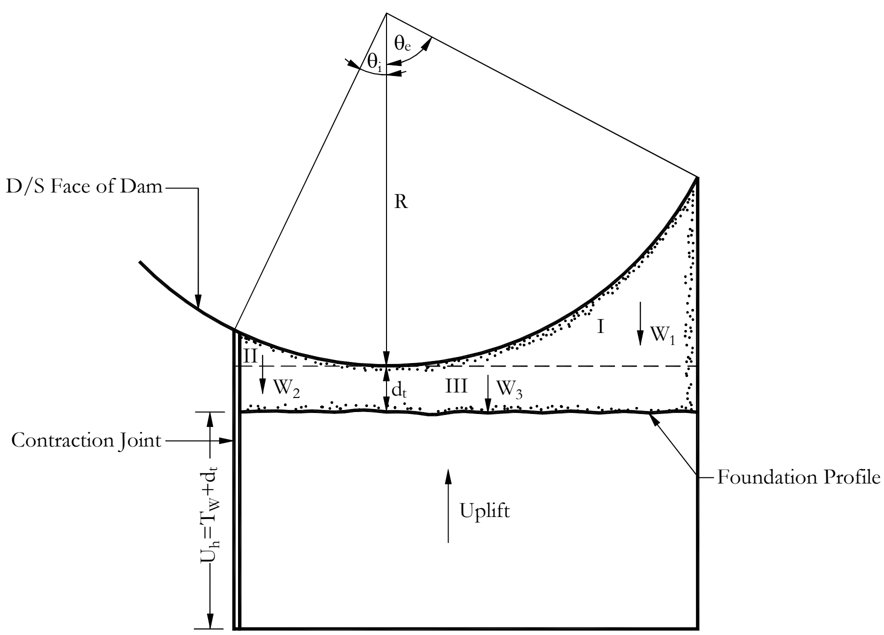


Figure 21: Uplift and body forces on solid roller bucket (IS: 11527 – 1985)

The required number of anchors shall be determined for F_u applying a factor of safety of 1.2 on the tensile yield strength of the reinforcing steel.

11.2.3.1 Length of Anchors

The length of the anchors shall be found such that the following criteria are satisfied:

a. The rock mass into which the anchors are grouted shall not be dislodged by the uplift pressure.
b. The bond stress between the steel and grout as well as the grout and rock generated by the actual force in each anchor shall be less than the corresponding permissible bond stress.

To satisfy the first criteria, the anchor shall be long enough to engage a mass of rock whose submerged weight is sufficient to withstand the net upward force acting on the anchor (Figure 22), thereby maintaining the stresses on the horizontal plane passing through the bottom of anchor bars within the permissible tensile stress of the rock. For this purpose, a cone of rock with a vertex angle of 45º shall be assumed to be bonded to the anchor bar. If the anchors are closely spaced and the cones of bonded rock overlap, the entire rock mass up to bottom of the anchors shall be taken as countering the uplift forces on the plane Z-Z (Figure 22).

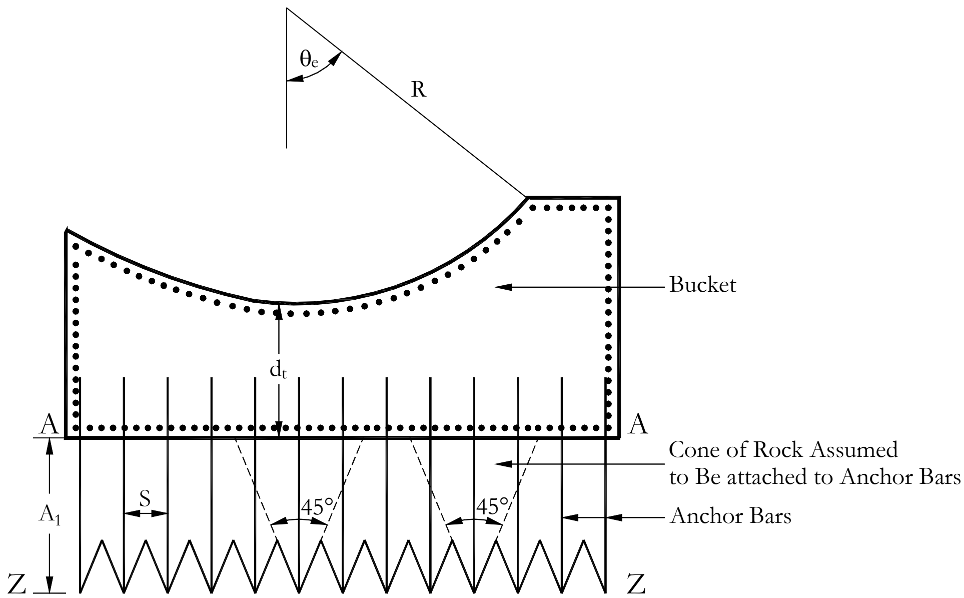


Figure 22: Anchor bars for countering uplift under solid roller bucket (IS: 11527 – 1985)

For checking the second criteria, the values of permissible bond stress shall be based on actual tests; however, in the absence of tests data, a grout proportion of 1:2 with permissible bond stress between steel and grout equal to 0.6 MPa and between grout and rock equal to 0.4 MPa may be assumed.

12. DESIGN OF DEFLECTOR BUCKETS

The design of a deflector bucket shall principally include the hydraulic and structural design of the bucket. However, selection of the location of the bucket and the jet impact shall be an important consideration in the design.

12.1 Site Selection

Selection of sites for the deflector bucket and the impact area shall be made in accordance with the recommendations provided in the following sections.

12.1.1 Deflector Bucket

The deflector bucket shall be founded on sound stable material with small settlement risk and substantial resistance to erosion. The foundation material shall be capable of resisting the high resultant force in the bucket due to the centrifugal slinging of the high velocity jet in it. It shall also be capable of resisting local foundation erosion due to water running down the end of the bucket at flows lower than the design flow.

12.1.2 Jet Impact Area

The jet emanating from the deflector bucket shall return to the river bed at a site where the consequences of potential erosion due to the jet impact are minor and acceptable from the point of view of safety of the diversion structure. To this end, this site shall be located away from the foundations of diversion structure and any hillside or slope where slope stability could be endangered if erosion becomes substantial.

For economy, the impact area shall have erosion-resistant bedrock so that a concrete base can be avoided. The rock bed shall be composed of large blocks so that the rock does not get ripped up by the pulsating pressures caused by the impact of the jet.

Where the impact area has to be located on an erosive rock bed, a preformed plunge pool with concrete or riprap base and lining shall be provided. Alternatively, erosion of the river bed may be allowed to take place until the plunge pool reaches an almost stable depth.

12.2 Hydraulic Design

A deflector bucket shall be designed to send the high velocity jet from the overflow section into the air in a trajectory formed for maximum dispersion through air contact before the water jet hits the ground again. The design shall include determination of the bucket shape, bucket invert elevation, radius or principal geometrical parameters of the buckets, lip elevation and exit angle, trajectory length and estimation of the scour downstream of the overflow section.

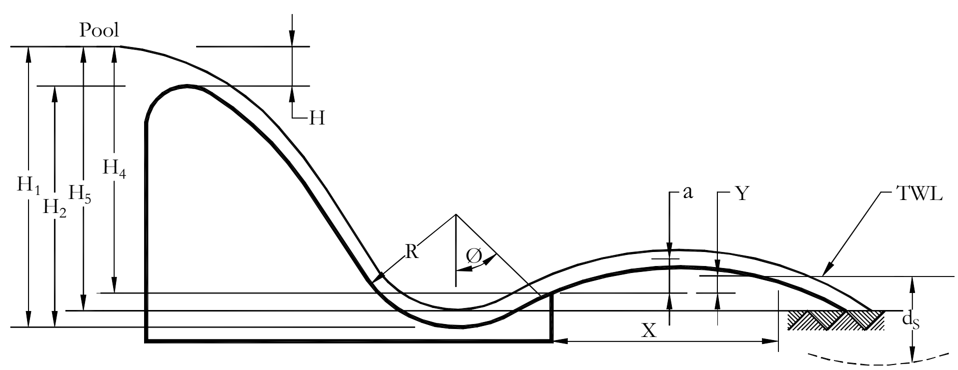


Figure 23: Definition sketch of deflector bucket

12.2.1 Bucket Shape

The bucket shape shall be fixed to achieve satisfactory trajectory height and length of throw in the flip action. From practical considerations, a preformed circular shape shall generally be preferred. Hence, only these types of buckets are covered in the guidelines.

12.2.2 Bucket Invert Elevation

The bucket invert elevation shall be fixed based on the site and tailwater conditions. For a clear flip action, the bucket lip shall be kept above the maximum tailwater level. In doing so, care shall be taken to ensure that the downstream geological conditions of the river bed and its abutments are sound enough to withstand the scour resulting from the impact of the jet.

For economy in construction, the deflector bucket may be placed near the natural bed level. This setting may also create a beneficial ground roller just below the end sill, thereby helping in piling up material against the end sill and reducing the possibility of erosion immediately downstream of the bucket. In such cases, the bucket invert level shall be fixed considering the following:

a. A minimum concrete cover of 1.5 m shall be available over the bed rock.

b. The bucket setting shall not submerge the bucket lip to an extent that the flip action of the bucket is transformed into roller action, thereby generating heavy sub-atmospheric pressures at the lip. The limiting submergence, D_4, beyond which this phenomenon may occur shall be determined from Figure 24. Generally, a submergence of more than 70 percent of the tailwater depth required for formation of a hydraulic jump may adversely affect proper performance of the deflector bucket. The safe maximum submergence may be assumed to be equal to the critical depth, D_c, over the lip elevation.

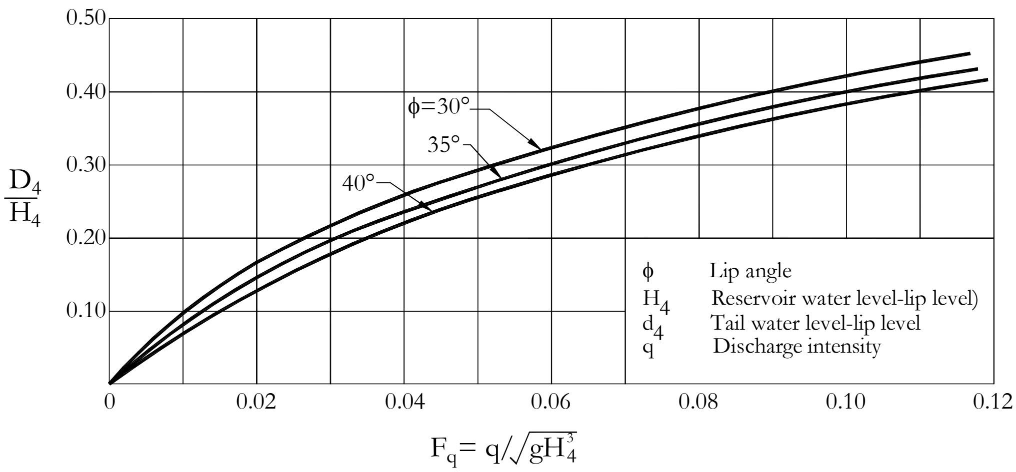


Figure 24: Limiting submergence between trajectory action and roller action (IS: 7365 –1985)

12.2.3 Bucket Radius

The bucket radius, R, shall be long enough to ensure that the streamline distribution of the flow in the bucket is not altered by the floor pressures. This condition shall be met to maintain concentric flow as the water moves around the bucket curve and to avoid the tendency of the water to spring away from the bucket.

For preliminary design, R may be obtained from the following relation (IS: 7365 – 1985):

Eq. 19
$$R = (0.6 - 0.8)\sqrt{H \cdot H_5}$$

where H_5 is the difference in the pool elevation and jet surface elevation on the bucket. The higher value of 0.8 shall preferably be used for preliminary design. However, the value of R thus obtained shall not be less than three times the maximum depth of flow entering the bucket, i.e. $R \geq 3D_1$.

The radius shall be finalized based on model tests by finding the actual pressure conditions existing in the bucket and avoiding hurdling conditions on the bucket.

12.2.4 Bucket Lip

The design of the bucket lip shall consist of fixing its angle shape and lip height.

12.2.4.1 Lip Angle

For clear flip action, the lip angle shall normally be adopted between 30° and 40º. Within this range, the choice of lip angle shall depend on the minimum throw permissible under the local rock conditions. Adoption of larger lip angles in this range shall increase the trajectory length and provide better dissipation as the jet impacts the tailwater at a steeper angle with less violent side eddies; however, steeper angle of impact may lead to deeper scour in the river bed.

For submerged lips, the lower lip angle of 30° may be adopted to minimize sub atmospheric pressures on the lip.

12.2.4.2 Shape

The top surface of the bucket lip shall be made flat if the tailwater level is lower than the lip level. Otherwise, the lip shall have a downstream slope of 1 in 10. In some cases, aeration may have to be provided. The need for aeration shall be finalized after model studies.

12.2.4.3 Height

The height of the bucket lip, h, shall be sufficient to prevent water from merely overriding the bucket lip instead of being turned and flipped out of the bucket. To effectively turn the flow, the bucket height shall be at least high enough to intersect the forward-projected slope of the water surface at the point of curvature of the overflow section and the bucket curve (Figure 23). In order to ensure that the flow follows the bucket curve and does not override the downstream lip, the bucket height shall not be less than the following (USACE, 1992):

Eq. 20 $$h_{\min} = R - R\cos(\alpha - \tan^{-1} S)$$

where

Eq. 21 $$\alpha = \tan^{-1}\left\{\frac{\sqrt{D_1(2R - D_1)}}{R - D_1}\right\}$$

describes the minimum deflection and angle S is the slope of the spillway chute adjacent to the bucket.

When $\alpha > \tan^{-1}S$ resulting in $h_{\min}=0$, the bucket height shall be defined by the required lip angle ϕ. A trial-and-error adjustment of the bucket radius and/or bucket lip angle may be necessary to meet or exceed $h_{\min}$ as defined in Eq. 20. The required height of the bucket lip, h, above the bucket invert necessary to satisfy the desired lip angle ϕ shall be determined by the following equation (USACE, 1992):

Eq. 22 $$h = R(1 - \cos\phi)$$

12.2.5 Bucket Termination

The deflector bucket shall terminate with a 90° cut from the bucket lip. Its side walls shall terminate at the lip to allow sufficient air to be drawn below the point of the trajectory separation from the bucket lip. Failure to allow sufficient air to the underside of the jet shall cause jet flutter with resultant pressure fluctuations and possible cavitation damage.

12.2.6 Alignment

The bucket shall be aligned such that its spreads the flow at the impact area across as much of the river channel as possible, thereby minimizing river bed adjustment and return flow from the downstream tailwater. The bucket may be aligned to direct the trajectory impact to a pre-selected location by curving or adding appurtenances to the bucket. In this case, model studies shall be performed to confirm the final design.

12.2.7 Trajectory Length

The trajectory length, X, of the jet (Figure 23) shall depend upon the bucket lip angle, the initial velocity of the jet and the difference in elevation between the lip and the tailwater. It shall be calculated using the equation (IS: 7365 – 1985)

Eq. 23
$$\frac{X}{H_V} = Sin\,2\phi + 2Cos\,\phi\sqrt{Sin^2\phi + Y/H_V}$$

where X is the horizontal throw distance from bucket lip to the centre point of impact with tailwater in m, Y is the difference between the lip level and tailwater level in m, taken positive for tailwater below the lip level, H_v is the velocity head of jet at the bucket lip in m and ϕ is the bucket lip angle with the horizontal in degrees.

The vertical distance of throw, *a,* above the lip level may be calculated from the following formula (IS: 7365 – 1985):

Eq. 24
$$a = \frac{v_a^2 \sin^2\phi}{2g}$$

where a is the vertical distance from the lip level to the highest point of the centre of jet in m, v_a is the actual velocity of flow entering the bucket in m/s and g is the acceleration due to gravity in m/s^2.

When Y is negative, model studies may be carried out to confirm the values of X and a.

Figure 25, which presents curves for trajectory lengths for lip angles between 0° and 45° based on Eq. 23, may be used to judge the point of impact of the jet. The actual trajectory length may be less than indicated depending upon the energy loss on the overflow section, air entrainment in the jet, etc.

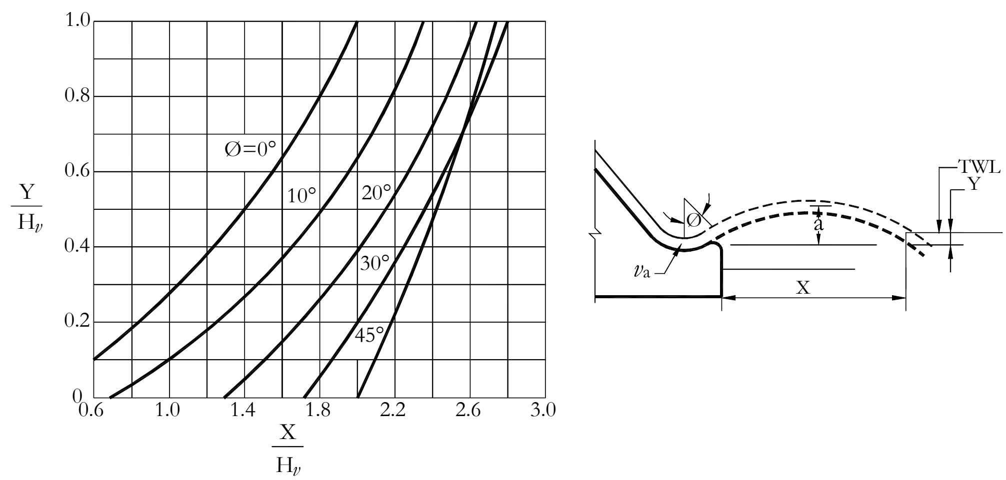


Figure 25: Trajectory lengths for deflector bucket (IS: 7365 – 1985)

12.2.8 Estimation of Downstream Scour

The scour below deflector buckets shall be governed by the combined effect of factors such as discharge intensity, height of fall, tailwater level, bucket lip angle, mode of operation of the overflow section, type and degree of homogeneity of rock, time involved in the process of scour, etc. However, the depth of scour, d_s, may be realistically evaluated considering two dominant factors, namely discharge intensity q and the total head H_4, through the following equation (IS: 7365 – 1985):

Eq. 25
$$d_s = m\sqrt{qH_4}$$

where d_s is the depth of scour in m, m is a constant taken as 0.36 for minimum expected scour, 0.54 for probable scour and 0.65 for ultimate scour, q is the discharge intensity in m^2/s and H_4 is the difference in reservoir pool elevation and bucket end sill elevation in m.

While selecting the appropriate value of m, minimum expected scour shall be taken as the minimum scour in any case. Similarly, the probable scour shall be taken as the scour which may reasonably be expected in any individual case of sustained operation of the overflow section, while the ultimate scour shall mean the final stabilized scour.

Alternatively, the probable scour may also be computed using the following relation (IS: 7365 – 1985):

Eq. 26
$$d_s = 1.9\, H_3^{0.225} q^{0.54}$$

where H_3 is the difference in reservoir pool elevation and the tailwater elevation in m.

12.2.9 Training Wall

The training walls for deflector buckets shall normally be extended about 10 m beyond the bucket lip to overcome the formation of return flow eddies, etc, and thereby avoid the entry of the downstream bed material into the bucket in the event of one or two of the end crest gates remaining unopened. It may be preferable to flare the training walls beyond the lip to provide aeration to the jet.

To account for the greater normal depth of the air-water mixed jet over that of clear water observed in prototypes, a freeboard, H_f, computed from the following empirical relation shall be provided to the wall (IS: 7365 – 1985):

Eq. 27
$$H_f = 0.61 + 0.0371 v_a D_1^{1/3}$$

where v_a is the actual velocity of flow entering the bucket in m/s and D_1 is the depth of flow entering the bucket in m.

The freeboard obtained from the above expression shall be adopted for preliminary design and shall be confirmed from model studies and detailed calculations.

The foundation of the training wall and/or lining below it may be taken down to the scour level as in discussed for roller buckets in Section 14.1.3.

12.2.10 Divide Walls

Divide walls provided for deflector buckets shall be designed based on the criteria defined in Section 11.1.6 for solid roller buckets.

12.2.11 Drainage

Subsurface drainage shall be provided under the bucket to reduce uplift pressures under it. In addition, the bucket surface shall be adequately drained to prevent water impoundment in it. Provisions for both types of drainage shall be made in accordance with details provided in Section 13.

12.2.12 Model Studies

Model tests on the deflector bucket shall be performed to ensure its satisfactory hydraulic performance. The test may include a model of the bucket itself to test its performance under different flows. Satisfactory performance of the bucket shall be judged by the trajectory height and length as well as existence of sub-atmospheric pressures on the bucket profile and lip.

Where the impact area of the jet lies in erodible foundation, the tests may also include a loose bed model to illustrate the potential erosion or the effect of alternative designs for the plunge pool. In the latter case, the test results shall be converted to prototype predictions considering the fact that the dissipation of the jet in the air is grossly underrepresented in the model and requires suitable adjustment.

12.3 Structural Design

Structural design of the deflector bucket shall conform to the stipulations in Section 11.2 for solid roller buckets.

12.3.1 Estimation of Bucket Pressure

Theoretical studies and model and prototype data indicate that bottom pressures change continuously throughout the bucket and are a function of the entering velocity and depth of flow, radius of curvature of the bucket and angle of deflection of the flow. The centrifugal force together with the corresponding water depth in the bucket would indicate the pressure on the bucket. The equation in terms applicable to trajectory bucket pressures can be written as (IS: 7365 – 1985):

Eq. 28

$$P = \gamma_w D_1 \left(\frac{v_a^2}{g\mathrm{R}} + 1 \right)$$

where P is the bucket pressure in kN/m^2, γ_w is the weight of water in kN/m^3, v_a is the actual velocity of flow entering bucket in m/s, g is the acceleration due to gravity in m/s^2, R is the radius of bucket in m and D_1 is the depth of flow entering bucket in m.

Figure 26 presents maximum theoretical bucket pressures for velocities of 10 to 45 m/s and R/D_1 ratios of 4 to 10. Actual pressures vary considerably over the bucket. However, the curves shown in Figure 26 shall be used to estimate the hydraulic load per square meter of the bucket surface. The design of training walls adjacent to the bucket shall also allow for these pressures. For actual variation of pressure along bucket profile and along the retaining walls model studies are necessary.

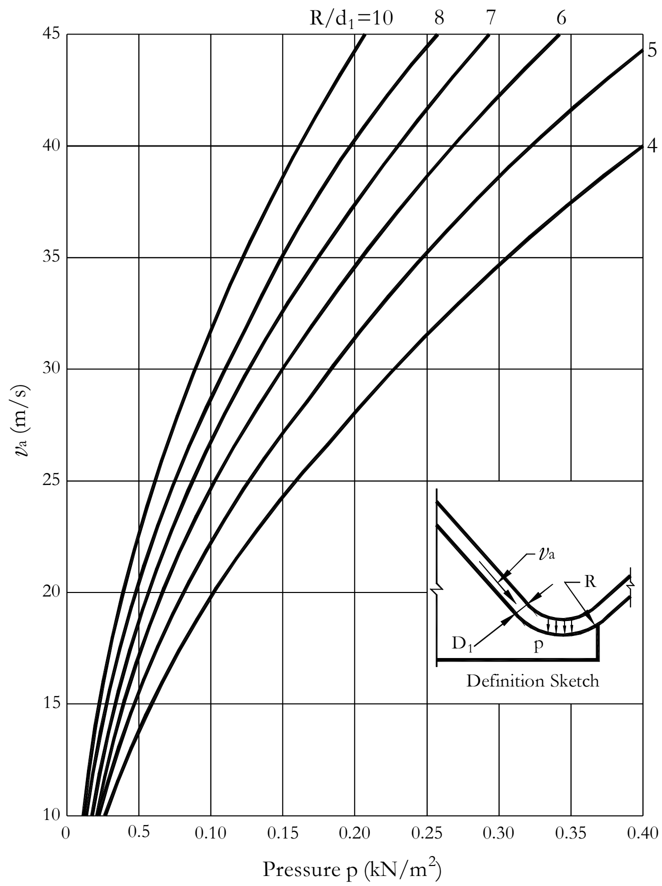


Figure 26: Maximum theoretical bucket pressures on deflector bucket (IS: 7365 – 1985)

13. DRAINAGE ARRANGEMENTS FOR ENERGY DISSIPATORS

Irrespective of their type, energy dissipators shall be provided with surface and subsurface drainage arrangements. The type of drainage arrangement shall depend on the head and tail water levels and the foundation strata.

13.1 Subsurface Drainage

Drainage shall be provided under energy dissipators to reduce uplift pressures and generally limit them to the required design limits. Even if grout curtains are provided under diversion structures to reduce seepage through the foundation, some means of subsurface drainage shall be provided to intercept water which may percolate through and around the grout curtain and which, if not removed, may build high hydrostatic pressures on the energy dissipator.

13.1.1 Design Considerations

Subsurface drainage systems shall be designed for the maximum probable uplift pressures and quantity of seepage water for foundation material under the most adverse head and tail water conditions, with or without groundwater. Ample factor of safety shall be used in the design.

13.1.2 Types of Drainage Systems

Depending on the nature of the diversion structure and the foundation material, drainage under stilling basin slabs may be provided in one of the following forms:

13.1.2.1 Vertical Formed Holes or Pipes

Vertical formed holes or pipes through floor slabs shall be provided for subsurface drainage of energy dissipators of minor headworks. The holes or pipes shall be spaced at 2 to 5 m in each direction (Figure 27).

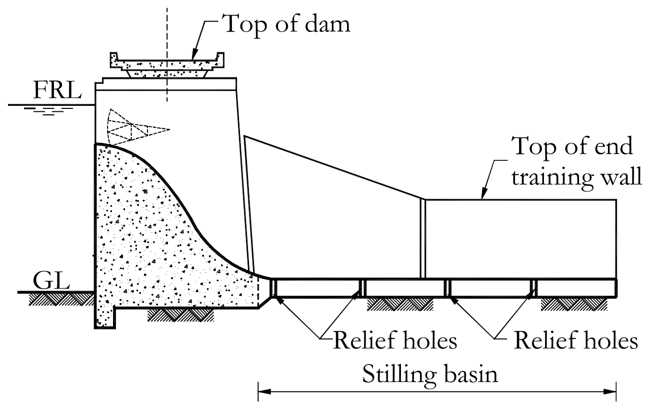


Figure 27: Relief holes or pipes in apron floor (IS: 11772 – 1986)

13.1.2.2 Half-round Pipe Grid Drains or Tile Drains

For low head diversion structures, subsurface drainage may be provided through a grid of half-round pipe drains or tile drains along the foundation surface (Figure 28). The grid shall follow the alignment of vertical or inclined drain holes, if provided, to facilitate collection and disposal of water percolating from them. The drains in the grid shall have a minimum diameter of 200 mm, and they shall lead the water to relatively larger collector half round drains, galleries or sumps near the upstream and/or downstream ends of the energy dissipator and/or near the training walls. These drains, galleries or sumps shall, in turn, convey the water to outlets provided on the downstream face of the overflow section and/or the upstream and/or the downstream face of the energy dissipators and/or the sides of the training walls. From the outlets, the water shall be released either by pressure or by gravity flow. The outlets shall be provided with non-return valves flush with the exit surface. For safety, each collector drain shall have at least two or preferably more outlets so that all drains may function satisfactorily even if some outlets get choked up.

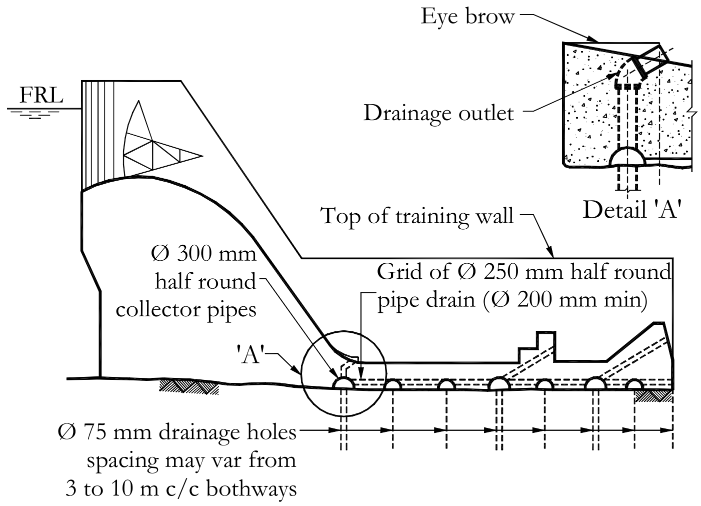


Figure 28: Drainage system with half round pipe drains or tile drains (IS: 11772 – 1986)

Alternatively, the grid of half round drains and/or large diameter collector half round drains shall be connected with galleries below the energy dissipators to lead water into sumps provided below the energy dissipators and/or on backsides of the end training walls. The drains may be connected directly to sumps as well. From the sumps, water shall be pumped for disposal from suitable locations above the maximum tail water level.

The half round drains may either be vitreous clay or plain concrete pipes. They shall be laid in graded material which acts as filter or on sub-grade as recommended in Section 13.1.3.

The drain holes shall usually be NX holes (75 mm diameter) drilled after completion of the foundation grouting in their vicinity, if any, to avoid their clogging. They shall be spaced at 3 to 10 m intervals in both directions, with closer spacing being adopted for rocks having low permeability. The depth of the drain holes may vary from 20 to 40 percent of the tail water depth when no supporting data is available; however, if details on the stratification and joints in foundation rocks, their permeability and their anticipated quantity of seepage are available, the depth of vertical or inclined drain holes shall be based on such data.

Vertical drain holes shall be adopted where joints and stratification in foundation rock have complex or dominantly horizontal pattern. Likewise, inclined drain holes shall be adopted where joints and stratifications in foundation rock have dominantly inclined pattern.

Horizontal drain holes shall be preferred where joints and stratification in foundation rock have a dominantly vertical pattern. These holes shall be drilled into foundation rock from the gallery either perpendicular to its alignment or in a fan shape with a gentle slope towards the gallery for easy disposal of percolating water. The depth of such holes may be up to 20 m or more depending on ease of drilling and subsequent maintenance for their effective functioning.

13.1.2.3 Drain Holes and Relief Wells

For overflow sections with higher heads on tight geological formations, subsurface drainage may be attained through drain holes drilled into the foundation rock in combination with formed holes or pipes through the floor slabs. In pervious foundation materials, relief wells may be used for this purpose.

13.1.3 Sub-grade

The half-round drainage pipes may be placed directly on foundation rock or on a sub-grade consisting of a porous concrete pad laid to required grades on foundation rock. The porous concrete may be used under the pipes to maintain proper connections among them and to maintain their levels and grades. Gravel blankets, wherever provided, shall be well graded to prevent movement of foundation material with flow of seepage water.

Arrangements for laying the half round drains may follow the alternatives shown in Figure 29.

13.1.4 Drainage Outlets

Seepage water collected in subsurface drains may be disposed off by gravity or pressure. Disposal under gravity flow may be achieved through one of the following methods:

a. Outlets through dentated sills for stilling basins (Figure 28).
b. Outlets from the downstream face of the sill of solid roller bucket (Figure 30).
c. Outlets from downstream face of sill of deflector bucket (Figure 31).
d. Eyebrow outlets on downstream glacis of spillway (Figure 28).

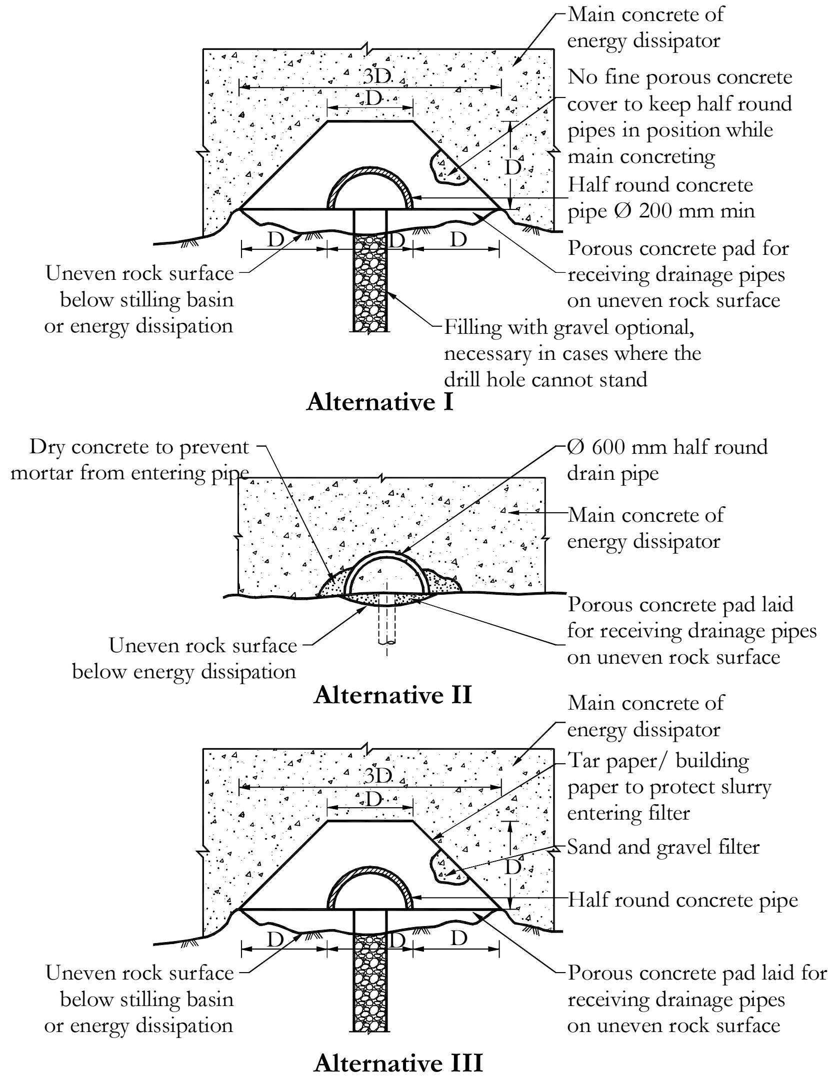


Figure 29: Alternative arrangements for laying half-round pipes (IS: 11772 – 1986)

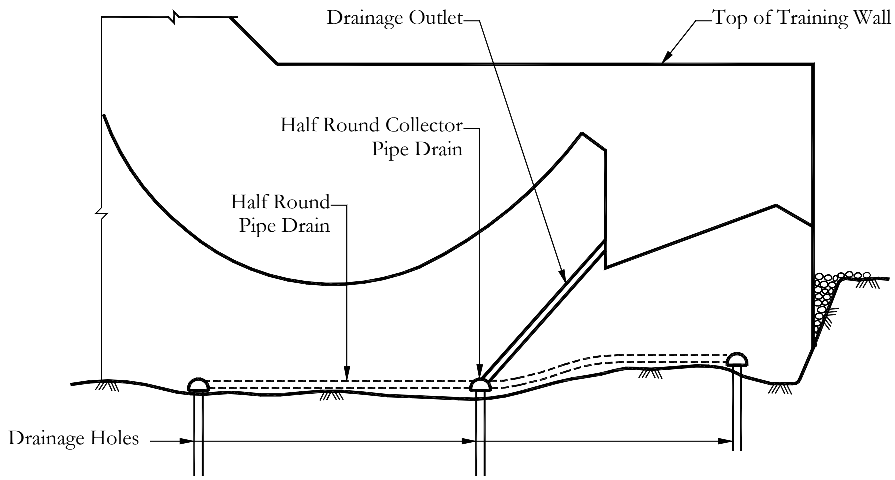


Figure 30: Drain outlet for solid roller bucket (IS: 11772 – 1986)

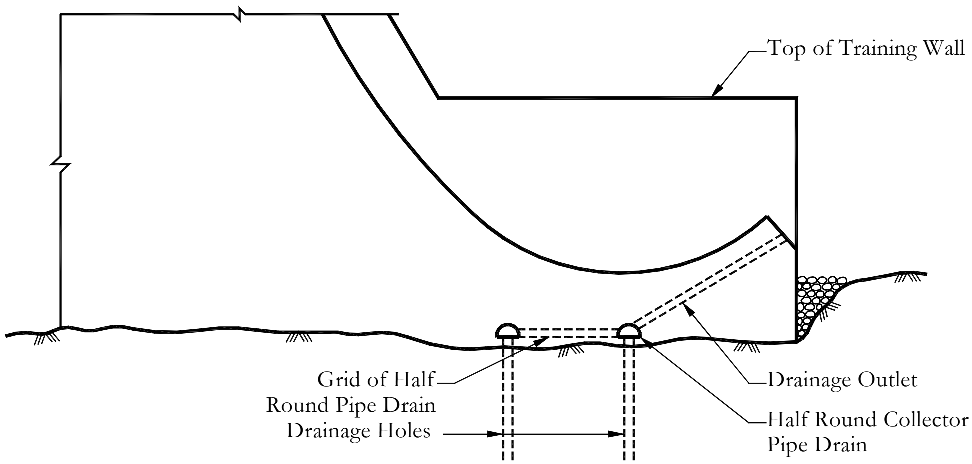


Figure 31: Drain outlet for deflector bucket (IS: 11772 – 1986)

For large stilling basins or buckets having operating heads higher than 50 m or inflow velocities greater than 30 m/s, drain outlets through the stilling basin apron or bucket floor shall not be provided as the resulting projections may trigger cavitation.

For disposal under pressure, the seepage water shall be pumped out from the sumps provided below the stilling basins or buckets anchor and/or behind the end training walls.

13.2 Surface Drainage

Due to potential for cavitation damage, floor drains shall be avoided for draining the surface of basin aprons or bucket floors. Water impounded on the apron or floor shall be drained laterally through sumps with streamlined profiles provided close to training walls in low-velocity areas of the apron or floor. Where practicable, drain pipes shall be provided to dispose off the standing water. In such cases, great care on detailing shall be observed to prevent cavitation.

14. TRAINING WALLS

For concrete diversion structures, downstream training walls may be provided to guide the flow from the overflow section into the downstream channel. These walls may be provided in the form of gravity walls, anchored walls, combination of gravity and anchored walls or as cantilever or counterfort walls.

14.1 Hydraulic Design

Hydraulic design of training walls shall include determination of its height, length and foundation level (Figure 32).

14.1.1 Height

In the bucket portion, the top of the training wall may be kept 1.5 m above the maximum water level in the bucket. If the wall abuts against an embankment, the top of the wall may be higher as required by the embankment profile.

In the surge portion, the height of the wall shall rise at least to an elevation h_s + l.5 m, where h_s is the maximum surge height determined from Figure 33. The distance of the surge, S_d, shall be obtained from Figure 34. The wall can be given a trapezoidal shape for economy as indicated in Figure 32.

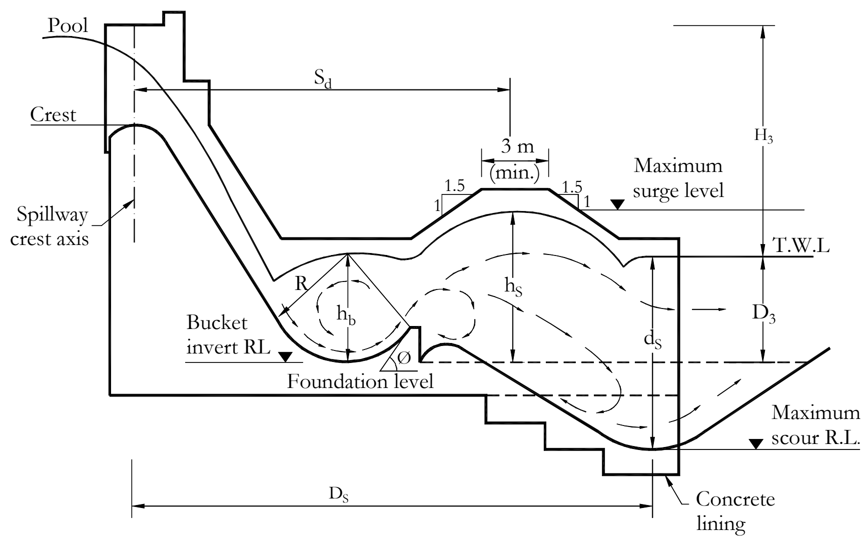


Figure 32: Definition sketch for top profile of training wall and scour and surge distances

Beyond the surge portion, the top of the training wall shall be kept at least l.5 m above the maximum tailwater level. If the wall abuts against an embankment, the top may be located at a higher level as required by the embankment profile. For preliminary design, the freeboard may be adopted based on the following empirical relationship (IS: 7365 –1985):

Eq. 29
$$\sqrt{H_f} = 0.0305(v_a + D_2)$$

where v_a is the actual velocity of flow entering the bucket in m/s and D_2 is the subcritical depth of flow in bucket including the boil height, etc. in m.

The freeboard thus computed shall be confirmed by model studies and detailed calculations.

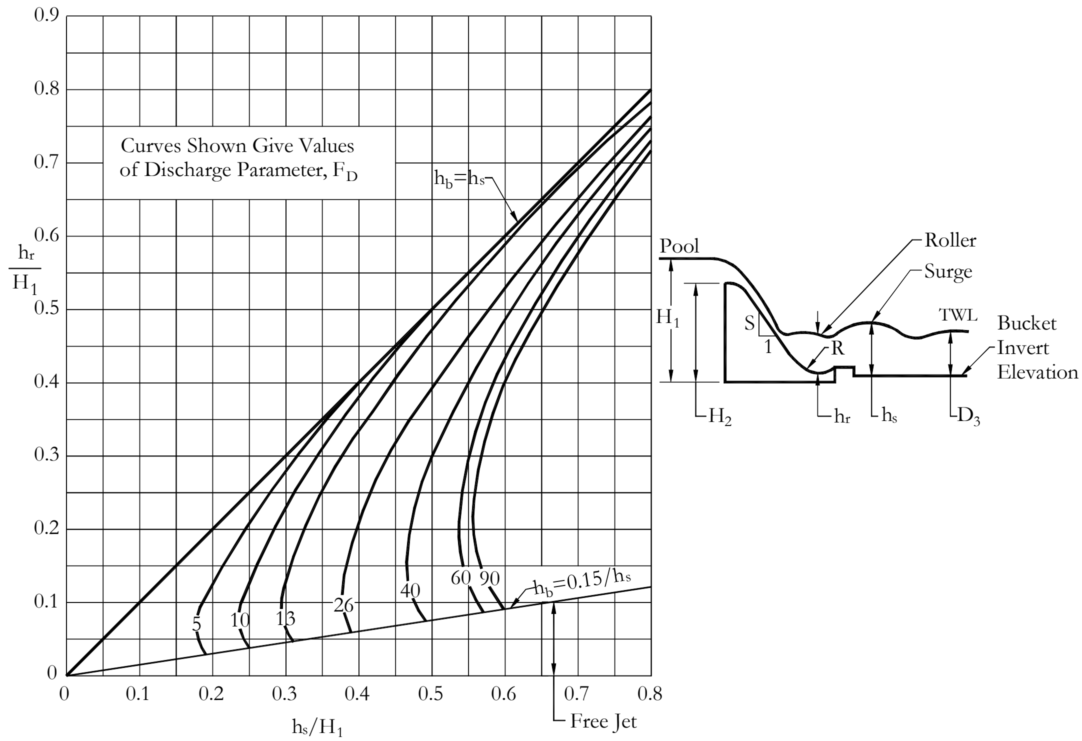


Figure 33: Curve for determination of surge height for solid roller bucket (IS: 7365 –1985)

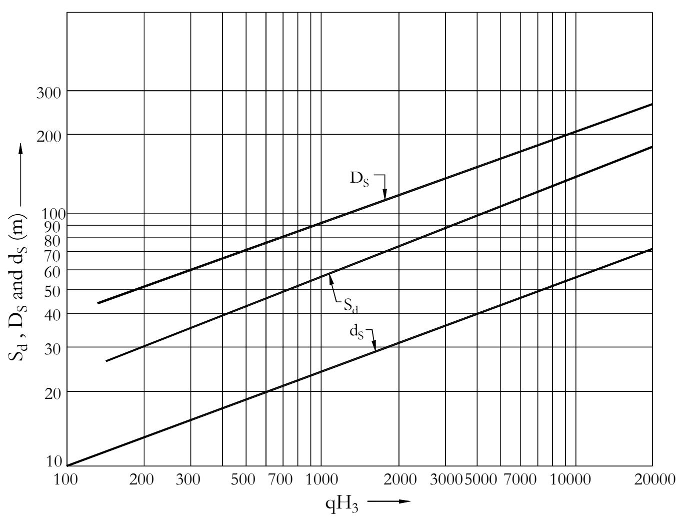


Figure 34: Training and divide walls scour depth and distance and surge distance (IS: 7365 – 1985)

14.1.2 Length

The length of the training wall shall generally be fixed to satisfy the following criteria:

a. The wall shall extend downstream beyond the surge portion by about 3 m or as dictated by the downstream toe of the adjoining embankment, if any.

b. If the toe of the embankment rests on erodible alluvium bed, the training wall shall extend beyond the toe at least by 15 m.

c. If the bed strata are erodible, the wall shall extend beyond the location of the deepest scour by about 5 m.

If required, a splay wall connecting the end of the training wall with the river bank may be provided to avoid erosion behind it.

14.1.3 Foundation

If competent foundation rock is not available up to the maximum scour level, the training wall foundation shall extend to a safe depth, generally about 2 m, below the maximum scour level. However, if competent rock is found above this level, the wall may be founded on the rock. In such cases, a concrete lining, about 1.0 to 1.5 m thick, shall be provided below the foundation level of the wall down to the anticipated scour level if the wall extends beyond the bucket.

For the above purposes, the maximum scour level shall be found from the scour envelope downstream of an overflow section obtained through model studies. In the absence of such studies, the maximum probable scour depth, d_s, and its distance, D_s, from the axis of the overflow crest may be estimated from Figure 34.

14.2 Structural Design

The following forces shall be considered in the design of training walls:

a. Dead loads.

b. Tailwater pressure and hydrodynamic loads due to flow, wherever applicable.

c. Uplift pressures.

d. Earthquake forces.

e. Earth pressures.

f. Live load or surcharge.

The training walls shall be designed for the following load conditions:

Condition	Loads
Normal	• Dead loads • No water on river side • Backfill submerged up to 50 percent and 33 percent of maximum tailwater elevation in case of earth backfill and rubble backfill, respectively, with drains effective • Full uplift
Severe	• Dead loads • No water on river side • Backfill submerged up to 50 percent and 33 percent of maximum tailwater elevation in case of earth backfill and rubble backfill, respectively • Full uplift • Earthquake forces
Sudden drawdown	• Backfill submerged up to maximum tailwater level • Water up to minimum tailwater level on river side with drains clogged • Full uplift
Spillway functioning	• Backfill submerged up to maximum tailwater level • Hydrostatic and hydrodynamic loads due to flow in the energy dissipator • Full uplift

The walls shall be designed such that no tensile stresses exist in them under normal loading conditions; however, nominal tensile stresses shall be permitted under other loading combinations. The maximum foundation pressures under the walls shall not exceed the safe bearing capacity of the foundation material. Furthermore, the walls shall have adequate factors of safety against sliding.

15. DOWNSTREAM PROTECTION

15.1 Block Protection

Pervious block protection shall be provided just beyond the downstream end of impervious floor as well. It shall comprise cement concrete blocks of adequate size laid over a suitably designed inverted filter for the grade of material in the river bed. The cement concrete blocks shall generally be not smaller than 1,500 × 1,500 × 900 mm size laid with gaps of 75 mm width and packed with gravel.

The length of downstream block protection shall be approximately equal to 1.5 time the design depth of scour below the floor level. Where this length is substantial, block protection with inverted filter may be provided in a part of the length, and block protection only with loose stone spawls shall be provided in remaining length. A toe wall of masonry or concrete, extending up to about 500 mm below the bottom of filter, shall be provided at the end of the inverted filter to prevent it from getting disturbed.

15.1.1 Graded Inverted Filter

The graded inverted filter shall roughly conform to the following design criteria (IS: 6966 – 1989):

$$\frac{D_{15}\text{ of filter}}{D_{15}\text{ of foundation}} \geq 4 \geq \frac{D_{15}\text{ of filter}}{D_{85}\text{ of foundation}}$$

where D_{15} is the grain size such that 15% of the soil grains are smaller than that particular size and D_{85} is the grain size such that 85% of the soil grains are smaller than that particular size.

The filter may be provided in two or more layers. Grain size curves of the filter layers and the base material shall be roughly parallel.

15.2 Loose Stone Protection

Beyond the block protection on the downstream side of a diversion structure located on permeable foundation, launching apron of loose boulder or stones shall be provided to spread uniformly over scoured slopes. The stone or boulder used shall not be less than 300 mm in size, and no stone shall weigh less than 40 kg. Where the stone is likely to be swept away due to high velocities or where somewhat smaller stones are to be used due to non-availability of stones of specified size, loose stone apron shall be provided in the form of cement concrete blocks or wire sausages of suitable size.

The quantity of stone provided at various locations of the headworks shall be adequate to cover the slopes of scour holes corresponding to the factors given in Table 1 for value of scour depth calculated from Lacey's formula. No allowance for concentration shall be made in the discharge per unit length in computing the normal scour for the protection works.

15.2.1 Slope of Launched Apron

The slopes of the launched apron shall primarily dependent on the grade and the size of the material in the river bed. For rivers in alluvium sandy reaches, the slope shall not be assumed steeper than 2:1 nor flatter than 3:1. The latter figure shall be adopted for river with very fine sand or silty bed.

15.2.2 Thickness of Material on Launched Slopes

The requirement of thickness of material on launched slope may be connected to grade of river bed material and river slope. The thickness of loose stone required for covering the launched slope shall be kept 1.25 τ, where τ is the value of thickness obtained from Table 4.

Table 4: Thickness of pitching

River bed material	River slope in m/km				
	0.05	0.15	0.20	0.30	0.40
Very course	400	500	550	650	700
Coarse	550	650	700	800	850
Medium	700	800	850	950	1000

(Source: IS: 6966 – 1989)

15.2.3 Length and Thickness of Laid Apron

The total quantity of loose stone worked out according to the above provisions shall be laid in a length of about 1.5 to 2.5 times design depth of scour below the floor level. The higher figure shall be adopted for flatter launched slope.

The thickness of stone at the inner edge shall correspond to the quantity required for a thickness. The material required for extra thickness of 0.25 on launched slope shall be distributed in the length of laid apron in the form of a wedge with increasing thickness towards outer edge.

16. PROTECTIVE PITCHING

Protection below energy dissipators shall usually be provided in the form of pitching, which may either be flat stone or needle rip rap, masonry lining or concrete lining. The type of the pitching to be provided and other details shall depend on several factors such as magnitude of velocities or wave height to be withstood, properties of the material such as angle of repose, chemical composition, etc, and the extent of surface to be protected. The type of the pitching to be provided may also be governed by the availability of pitching material. The stone pitching may be with wire net caging also, depending on the site requirements. The design velocity or wave height shall be determined from hydraulic model studies.

The following guidelines may be followed while designing the protective pitching.

a. The bank or the surface to be protected shall have, as far as possible, straight alignment or smoothest possible, straight alignment or smoothest possible curves as against highly curved or tortuous alignment.

b. The slope of the bank to be pitched shall be flatter than the angle of repose of the material forming the bank. For very large heights, intermediate berms may also be provided.

c. When the pitching is in the form of layers of hand placed or dumped dry stones, the weight and size of individual stone shall be determined from the following relationships (IS: 13195 – 1991):

Eq. 30
$$W_{50} = \frac{\pi V^2 \gamma_w^3 \gamma_p}{6C^6 (2g)^3 (\gamma_p - \gamma_w)^3}$$

Eq. 31
$$D_{50} = \left[\frac{6W_{50}}{\pi \gamma_p}\right]^{1/3}$$

where W_{50} is the weight of the stone in kg, with 50 percent material containing stones of weight W_{50} or less, V is the velocity of flow, m/s, γ_w is the specific weight of water, kg/m^3, γ_p is the specific weight of pitching material, kg/m^3, C is 0.86 for high turbulence level flows (stilling basins, etc) or 1.2 for low turbulence level flows (river closures, etc) and D_{50} is the spherical diameter of stone having the same weight as W_{50}.

d. If the embankment is subjected to waves in addition to flow velocity, the weight of the stone shall also be checked from the following relationship

Eq. 32

$$W = \frac{K}{(\mu\cos\alpha - \sin\alpha)^3} \cdot \frac{H^3\delta_P}{\left(\dfrac{\delta_P}{\delta_W} - 1\right)}$$

where K is a constant whose value is 0.43, μ is a constant with a value of 2.38, δ_P is the density of pitching material, H is the wave height, δ_p is the angle of slope of embankment and δ_W is the density of water.

e. The larger of the two values of W as calculated from Eq. 30 and Eq. 31 shall be adopted.

f. For high velocities, when the weight required of the individual stone is too large, wire net tied crates of several stones, of the required weight and size could be used. In such a case, the least dimension of the smallest stone used shall be about two times the opening in the wire net.

g. When the weight required is extremely large so that it could not be met with from stones or stone-crates, continuous lining of the surface by masonry or concrete could be resorted to. For such linings, two important factors to be decided, the size of the individual panel (to be cast as monolith) and the thickness of the lining. The following procedure shall be adopted:

h. Using the appropriate relationships decide the weight of the panel required against the velocity of the flow.

i. Using appropriate value of the specific weight of the pitching material r_P, (stone masonry or concrete), calculate volume of the panel.

j. Calculate the thickness of the lining using the formulae

Eq. 33

$$d = \frac{r_W}{r_P} \cdot \frac{V^2}{2g}\sqrt{\frac{1+S^2}{S}}$$

where d is the thickness of the lining, r_W is the specific weight of water, r_P is the specific weight of the pitching material, V is the velocity of flow, g is the gravitational constant and S is the slope of the bank expressed as S (Horizontal):1 Vertical.

k. Knowing the volume of the panel and thickness of the lining, the linear dimensions (i.e. size) of the panel could be worked out. This size may be proportioned to the overall size of the area to be pitched

l. The toe of the pitching shall be keyed down firmly into the bed rock. The depth of keying shall depend upon the quality of bed rock and the overburden. Where bed rock is not available at all or available at very large depth, the pitching shall continue in to the ground, along the same slope, adequately below the expected depth of scour, which shall be determined from the model studies.

m. For all types of pitching, adequate drainage to relieve the uplift pressure below the pitching, by way of weep holes, filters, etc, shall be provided

PART 2D – EMBANKMENT DAMS

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Part

2D

Embankment Dams

1. PURPOSE

Part 2D of the *Design Guidelines for Headworks of Hydropower Projects* provides technical criteria and guidance for design of embankment dams for headworks of run-of-river hydropower projects in Nepal. The guidelines are intended to ensure safe and economical design of these structures with due consideration of relevant issues, particularly those arising from conditions typical to Nepal.

2. SCOPE

The guidelines cover the design of embankment dams for use in run-of-river hydropower projects in Nepal. They include both earthfill and rockfill dams.

The guidelines presented in this part cover the hydraulic design of embankment dams on permeable and impermeable foundations. They discuss different types of embankment dams, their selection criteria, design considerations and data, embankment profiles, slope stability and strengthening, drain and anti-seepage arrangements, settlement, etc. Seismic analysis of embankment dams is also covered in the guidelines.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Axis of dam	Vertical plane or curved surface, chosen by a designer, appearing as a line in plan or in cross-section to which the horizontal dimensions of the dam are referenced.
Berm	A nearly horizontal step in the sloping profile of an embankment dam.
Composite earth dam	Earth dam consisting essentially of an inner or enclosed impervious section supported by two or more outer sections of relatively pervious material.

Dam crest	Upper (top) part of the dam body.
Embankment dam	A dam with the main section composed principally of gravel, sand, silt and clay. It is also called earthen dam.
Homogeneous earth dam	Earth dam composed of a single type of material, except for protective material on the exposed faces.
Length of dam	The length along the top of the dam.
Rockfill dam	Dam composed of rock, either dumped in the lifts or compacted in layers ,as a major structural element.
Rock toe	The downstream toe of an earth dam or other structure constructed of rock materials.
Seepage	Interstitial movement of water that may take place through a dam, its foundation or its abutments
Toe drain	Drain constructed at the downstream toe of an earth dam to collect and drain away the seepage through the dam and its foundation.

4. DESIGN OBJECTIVES

An embankment dam shall be designed to create a hydraulically sufficient structure, using locally available earth or rock, for retaining and diverting water for hydropower generation.

5. SCOPE OF DESIGN

The objectives of Section 4 shall be attained through proper hydraulic and geotechnical design of the embankment dam. Generally, the design shall entail:

a. Selection of the type of embankment dam.
b. Determination of embankment profiles.
c. Hydraulic design of drains and filters, anti-seepage arrangements, etc.
d. Stability analysis of embankment slopes.
e. Settlement studies.
f. Seismic analysis of embankments.

These activities shall be performed in accordance with the design principles and procedures discussed in the following sections.

6. DESIGN CRITERIA

Embankment dams shall be designed to meet the following criteria:

a. It shall be safe against excessive overtopping by wave action especially during high design flood flows.
b. The embankment slopes shall be stable during all conditions of the reservoir operations, including rapid drawdown, if applicable.
c. Seepage flow through the body of embankment dam, foundation and abutments shall be controlled so that no internal erosion (piping) takes place and there is no sloughing in areas where seepage emerges.
d. The embankment dam shall not overstress the foundation.
e. Slopes of the embankment dam shall be acceptably protected against erosion by wave action and from gullying as well as scour against surface runoff due to rain.
f. The embankment dam, foundation, abutments and reservoir rim shall be stable and shall not develop unacceptable deformations under earthquake conditions.

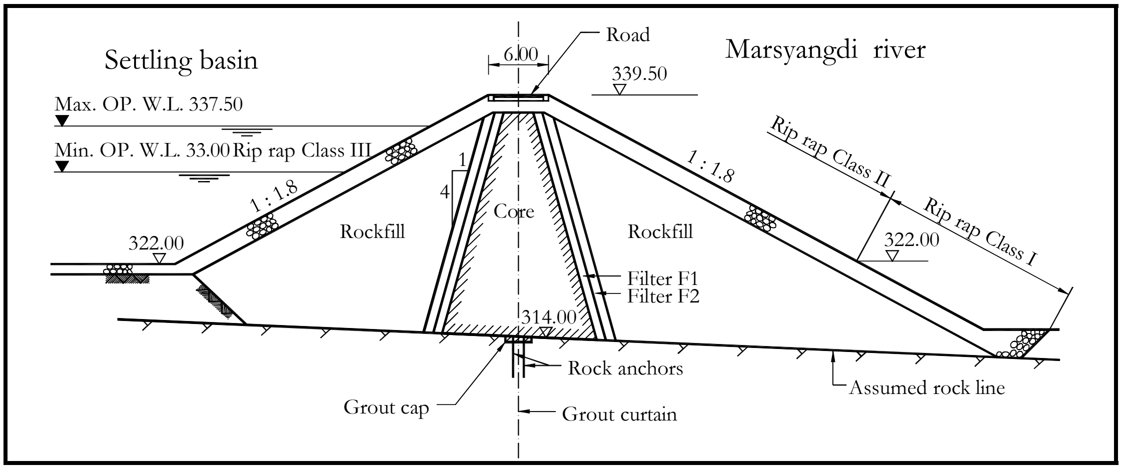


Figure 1: Embankment dam at Marsyangdi Hydroelectric Project, Nepal (NEA, 1983)

7. TYPICAL EMBANKMENT DAM PROFILE

Embankment dams shall have a trapezoidal cross-section composed of soil. It shall typically be composed of the following elements (Figure 2):

a. Dam body.

b. Dam crest.

c. Upstream and downstream berms.

d. Upstream and downstream lining.

e. Drains.

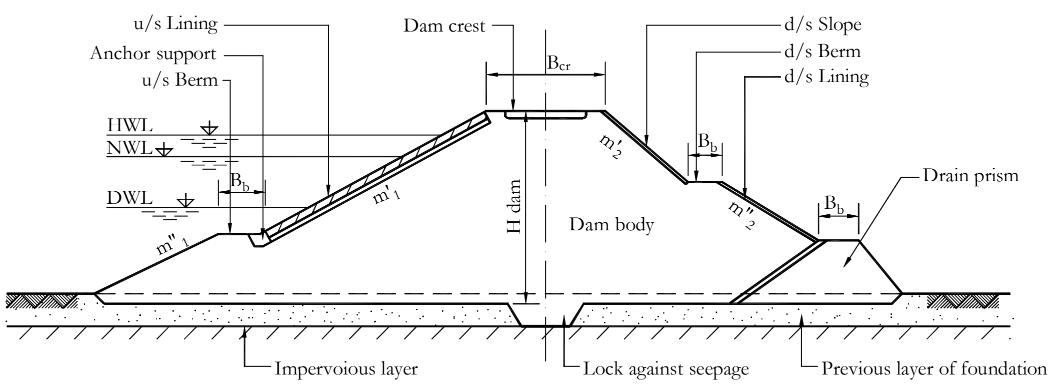


Figure 2: Cross-section of earthen dams

8. TYPES OF EMBANKMENT DAMS

Based on the structural composition of their dam body, embankment dams may belong to the following categories (Figure 4):

a. Dam with homogenous soils.

b. Dam with non-homogenous soils.

c. Dam with synthetic screen materials.

d. Dam with inclined core wall.

e. Dam with central core wall.

f. Dam with diaphragm.

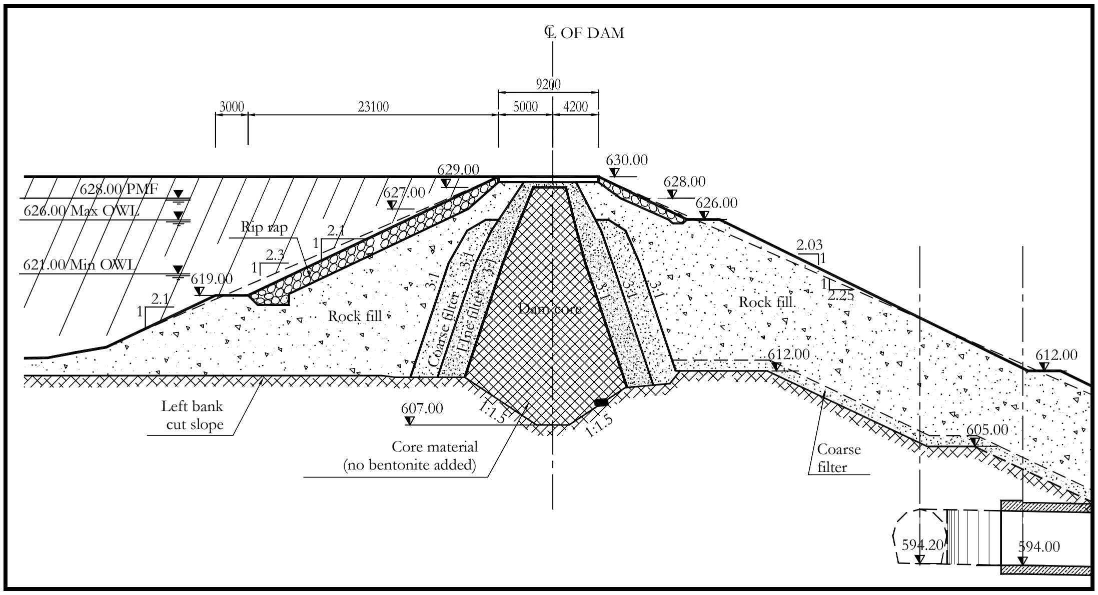


Figure 3: Embankment dam at Middle Marsyangdi Hydroelectric Project, Nepal (courtesy: NEA)

9. SELECTION OF TYPE OF EMBANKMENT DAMS

The type of embankment dam for a particular site shall be selected based on the following considerations:

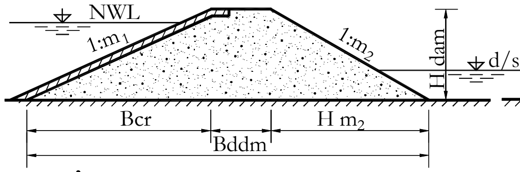
a. Engineering geological, hydrogeological and climatological conditions of the site.

b. Local availability of construction materials.

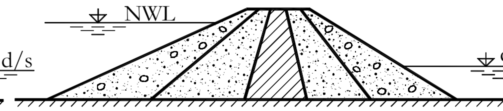
c. Availability of construction facilities and components of structures of the headworks.

10. DAM PROFILE

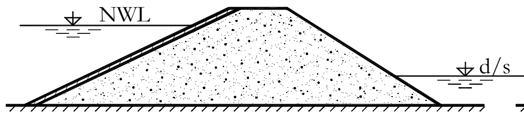
In the low head dams, slope coefficient shall be constant. In medium head dams, the slope will be variable, with its upper part having a higher slope. Generally, the slope coefficient shall be increased by 0.10 to 0.25 for each 10 to 15 m of dam height.



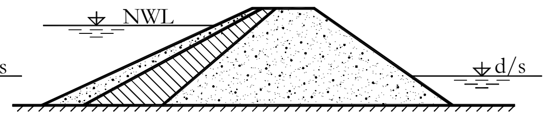
i. Dam of homogenous soil



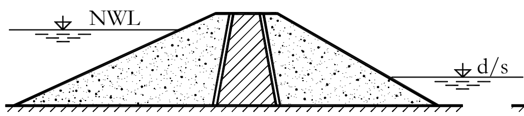
ii. Dam of non-homogenous soil



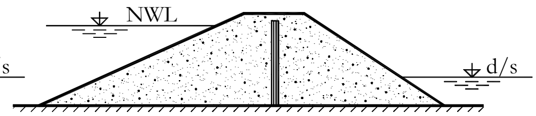
iii. Dam with sheet of synthetic materials



iv. Dam with inclined core



v. Dam with central core



vi. Dam with diaphragm

Figure 4: Types of embankment dams (Zhurablov, 1975)

For slope stability of small embankments, slope coefficients may be adopted from Table 1.

Table 1: Approximate values of embankment slope coefficients (m) of embankment dams

Soil in dam body	**Dam height, (in metres)**					
	≤ 5		5 to 8		8 to 15	
	u/s	d/s	u/s	d/s	u/s	d/s
Homogenous dams without drain:						
Clayey silty soils	2	2	2.5	2	3	2.25
Silty soils	2.5	2	2.5	2	3	2.5
Dams with clayey silt screen (cover) without drain:						
Sandy soils	2.25	2	2.5	2	3	2.5
Dams with clayey silty core without drain:						
Sandy soils	3	2	3	2	3.25	2.5

(Source: Nayak, 1993)

In order to reduce the earth volume, slopes with variable slope coefficients, with higher values near the foundation and lower values near the dam crest, shall be adopted. The slope bends in height shall be given with 10 to 15 m, taking change by 0.5 for upstream slopes and by 0.25 for downstream slopes.

Dimensions of the dam crest are determined by category of the road and its type (Table 2). If heavy vehicles are not considered, the minimum width of the dam crest may be as 3 m for low and medium head dams and 6 m for high head dams (Figure 5).

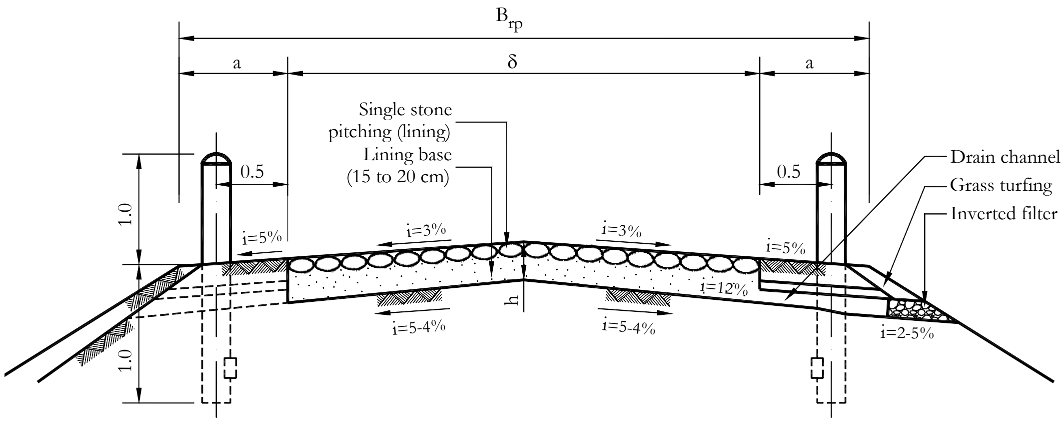


Figure 5: Arrangement of road at dam crest (Nayak, 1993)

Table 2: Dimensions of crest width of dam for roads

Characteristics of road	**Category of road**				
	I	II	III	IV	V
Width of dam crest, m	≥ 27.5	15	12	10	8
Width of runway, m	≥ 15	7.5	7.0	6.0	4.5
Width of footpath	3.75	3.75	2.5	2.0	1.75

(Source: Zhurablov, 1975)

Berms are provided at the slope of dam for supervision and maintenance of the slope covered, improvement of its stability and discharge of water from downstream slope. At the downstream slope, the berms are arranged usually at the places where slope coefficients are changed, i.e. at 10 to 15 m. In the absence of vehicle passing on them, the berm width are taken to be 1.5 to 2.0 m from the condition of location of its support.

11. FREE BOARD

This is considered to be a marginal height above the normal water level (NWL) for taking care of the high flood level (HFL), wave of the blowing wind and constructive safety margin depending upon the depth of reservoir. It is determined as

Eq. 1
$$h = h_c + \Delta h + a$$

where h_c is the maximum height of coastal wave on the embankment slope of the earthen dam, to be determined by Eq. 2, Δh is the height of the wind pileup to be determined by the Eq. 3 and a is the safety margin equal to 0.4 for the reservoir depth up to 25 m, 0.5 m for reservoir depth the same from 25 to 70 m, 0.70 m for reservoir depth the same from 70 m to 100 m and 1.0 m for the reservoir depth of more than 100m.

Eq. 2
$$h_c = \frac{2k_r}{m_1} h \sqrt[3]{\frac{\lambda}{h}}$$

where k_r is the coefficient to be taken from Table 3, m_1 is the embankment slope coefficient at the place where coastal wave takes place, h is the height of coastal wave to be determined by λ where it is the length of coastal wave to be determined by :

Table 3: Values of coefficient k_r

Types of embankment slope lining	k_r
Smooth	1.00
Concrete	0.90
Stone bridged	0.75 - 0.80
Stone thrown	0.60 - 0.75
Irregular stone	0.55

Along with coastal wave the wind pileup on the embankment slope also takes place. The height of wind pileup depends upon the wind velocity W_{10} at the height of 10 m above the reservoir water surface, fetch length of the reservoir D, depth of the reservoir H_1, and is determined by the formula:

Eq. 3
$$\Delta h = k \frac{W_{10}^2 D}{g H_1} \cos\alpha$$

where k is an empirical coefficient equal to $2x10^{-6}$, α is the angle formed between the reservoir water surface and wind direction and g is the acceleration due to gravity.

12. STRENGTHENING OF EMBANKMENT SLOPES

12.1 Lining of Upstream Slope

For the selection of type of slope pitched, it is essential to consider materials like stones, concrete and R.C.C. covers. Further it is necessary to take into account that for pitching with stones the local materials are used, work on it can be done throughout a year and its ductility (flexibility) allows it to preserve its property under the deformation of the dam body.

Pitching by stone is of two types: throwing and bridge pitching. This with stone throwing is presented in , which is as a rule, done from unsorted stones. Mass of the separate stones stable against wave action, is determined by the formula (Zhurablov, 1975):

Eq. 4
$$M_S = \eta\,\mu \frac{\rho_s \rho_w h^2 \lambda}{(\rho_s - \rho_w)\sqrt{1+m^3}}$$

where η is a safety factor equal to 1.5 for throwing of sorted stones and 2 for throwing of unsorted stones, μ is a coefficient equal to 0.025, ρ_S and ρ_W are the volume weights of stone and water, respectively, in kg/m^3, m is the slope coefficient of the embankment and λ is the length of wave in m.

The elements of wind wave are determined from natural observations, empirical formulae, diagram and nomograms. For low head dams, and for preliminary designs of dams having medium height the elements can be obtained from Table 4.

The wave elements on Table 4 are given for deep parts of the reservoir, for which the depth of water does not effect on the wave formation. For transferring of these values to the elements of wave on the low depth parts the following relationships are used (Zhurablov, 1975):

Eq. 5
$$h = h_0 \beta$$

and

Eq. 6 $$\lambda = \lambda_0 \alpha$$

where h and λ are the height and length of the wave, respectively, on the low depth of the reservoir, h_0 and λ_0 are table values of wave elements, respectively, for the deep parts of the reservoir and α and β are coefficients considering effect of the low depth of reservoir from Figure 6.

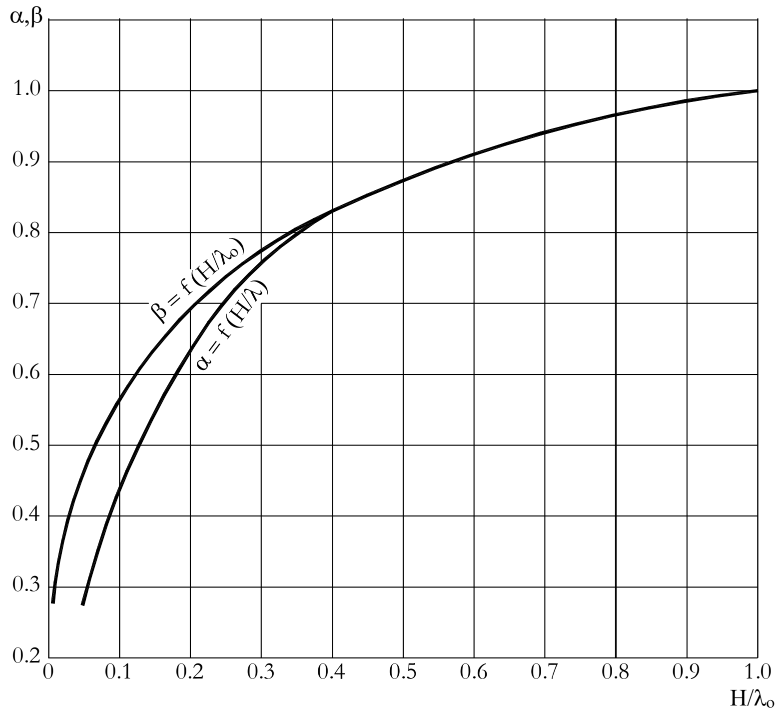


Figure 6: Coefficients α and β vs. H/λ_0 (H is depth of reservoir) (Zhurablov, 1975)

Thickness of lining by stone throwing is adopted equal to be $3D_{sph}$, where D_{sph} is the diameter of the stone sphere, to be determined roughly by the formula:

Eq. 7 $$D_{sph} = 0.9\sqrt[3]{M_s}$$

Bridge lining for pitching of upstream slopes of earthen dams is done in the same way as in the road slope covers. It may be of one or more layers. Form of stones must be with ratio of sides upto 1:2. Stones with length side are placed normal to the embankment slope. Single bridge lining is shown in Figure 7.

Thickness of stones (normal to slope) stable against wave action, can be obtained by the formula (Nayak, 1993):

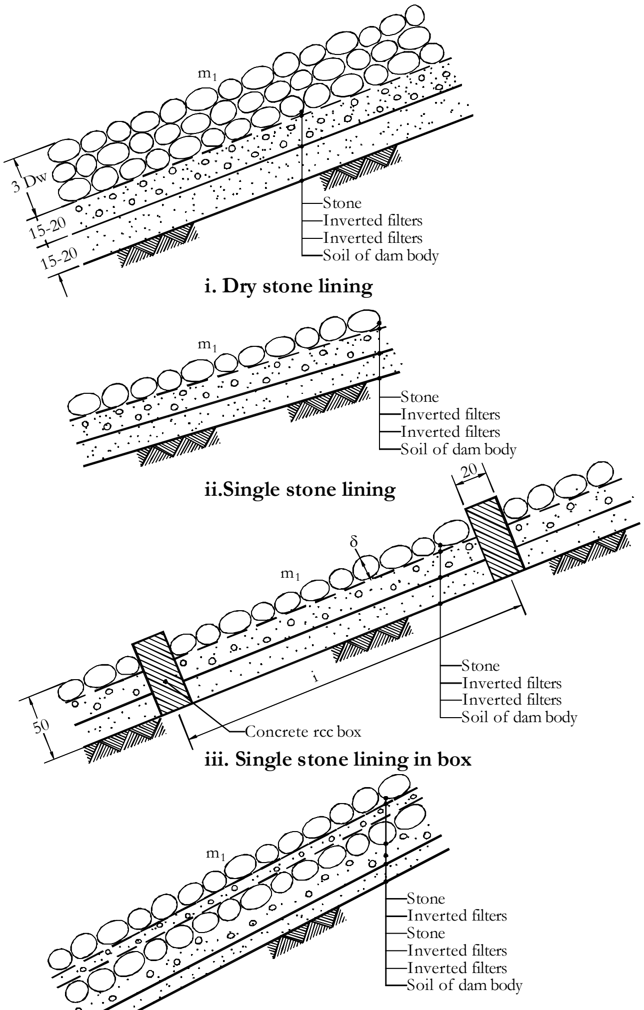
Table 4: Magnitude of elements of wind wave, m

Fetch length D, km	Slope of wave															
	1/10		1/12		1/13		1/14		1/15		1/17		1/18		1/19	
	Design velocity of wind, W10, m/sec															
	6		8		10		12		14		16		18		20	
	h_0	λ_0	h_0	λ_0	h_0	λ_0	h_0	λ_0	h_0	λ_0	h_0	λ_0	h_0	λ_0	H_0	λ_0
0.2	0.12	0.62	0.16	0.90	0.18	1.18	0.21	1.47	0.23	1.77	0.26	2.46	0.28	2.48	0.30	2.85
0.5	0.19	0.99	0.23	1.42	0.28	1.86	0.33	2.32	0.36	2.80	0.40	3.42	0.43	3.93	0.47	4.49
1.0	0.27	1.39	0.32	2.01	0.40	2.64	0.46	3.29	0.52	3.95	0.56	4.83	0.61	5.56	0.68	6.36
2.0	0.37	1.97	0.45	2.84	0.55	3.72	0.64	4.66	0.72	5.58	0.78	6.84	0.85	7.85	0.83	9.00
4.0	0.49	2.78	0.61	4.02	0.75	5.26	0.88	6.60	1.00	7.92	1.08	9.65	1.18	11.11	1.28	12.72
6.0	0.57	3.40	0.73	4.91	0.89	6.45	1.05	8.05	1.19	9.70	1.30	11.80	1.42	13.65	1.56	15.68
8.0	0.62	3.94	0.79	5.68	0.99	7.45	1.18	9.32	1.34	11.20	1.46	13.68	1.60	15.70	1.75	18.02
10.0	0.66	4.40	0.85	6.36	1.07	8.32	1.28	10.42	1.46	12.50	1.59	15.28	1.76	17.60	1.93	20.18
15.0	0.74	5.40	0.96	7.80	1.21	10.40	1.47	12.78	1.68	15.30	1.85	18.70	2.06	21.55	2.26	24.67
20.0	0.80	6.30	1.00	9.00	1.25	11.30	1.60	14.60	1.80	17.50	2.10	21.60	2.30	24.90	2.50	28.50
40.0	0.90	8.80	1.40	12.80	1.50	16.60	1.90	20.90	2.10	24.50	2.40	31.20	2.70	35.00	3.10	41.40
60.0	1.10	10.80	1.45	15.60	1.70	20.40	2.10	25.50	2.40	30.60	2.80	38.40	3.00	42.70	3.40	49.30
80.0	1.20	12.50	1.50	18.00	1.90	23.60	2.30	30.40	2.50	35.40	2.90	44.50	3.20	49.50	3.60	57.00
100.0	1.40	13.90	1.70	21.10	2.10	26.30	2.50	33.00	2.80	39.60	3.00	48.20	3.40	55.70	3.80	63.80

(Source: Nayak, 1993)

Eq. 8 $$\delta = 1.7h \frac{\rho_w}{\rho_s - \rho_w} \cdot \frac{\sqrt{1+m^2}}{m(m+2)}$$

where h is the height of wave in m.



iv. Double stone lining

Figure 7: Pitching of upstream slopes with stones (Zhurablov, 1975)

Concrete or R.C.C. lining is done in the form of plate (block), the dimensions of which are not standard, but in recent year there occurs a tendency to increase their area. Thickness of this plate is to be determined by the formula (Zhurablov, 1975):

Eq. 9 $$\delta = 0.07\,\eta\, h \frac{\rho_w}{\rho_p - \rho_w} \sqrt[3]{\frac{\lambda}{B}} \cdot \frac{\sqrt{1+m^2}}{m}$$

where B is the plate dimension in m, η is a coefficient taken as 1 for monolithic plates and 1.1 for assembled plates, ρ_p is the volume weight of plate in kg/m^{3}and λ is the wave length in m.

Note: Monolithic R.C.C. lining is done by the plates to be casted at the place with size on plan from 5 x 5 m to 20 x 20 m and more and thickness 0.15 to 0.50 m. Plates are separated from each other with deformation joints, which may be open or closed.

12.2 Strengthening of Downstream Slope

In the earth dams the downstream slopes are subjected to the process of natural weathering. The unprotected slopes are any way deformed due to this action. With the use of local materials the downstream slopes are strengthened for protection from their possible deformations. Character of strengthening should correspond to the main type of action.

Simple and cheap method of strengthening is considered to be bio-engineering. For clayey and sandy soils in the body of dam, a layer of the soil with thickness of 6 to 9 cm is filled, in which grasses can grow, for their rapid growth. Bio-engineering for strengthening of the embankment slopes are adopted there, where favourable conditions for grass growing prevails or periodical irrigation is conducted in the hot season. In these areas the slopes are

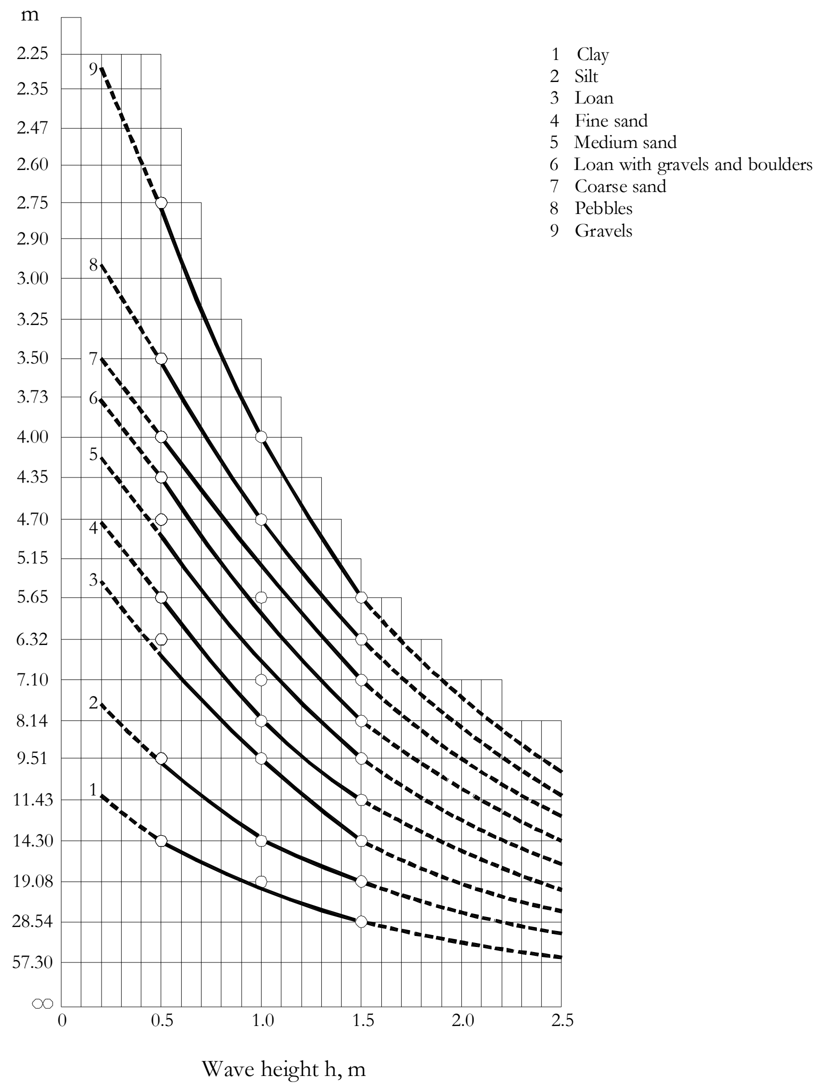


Figure 8: Diagram for determination of downstream unerodible slope of embankment dams (VNIIG, 1976)

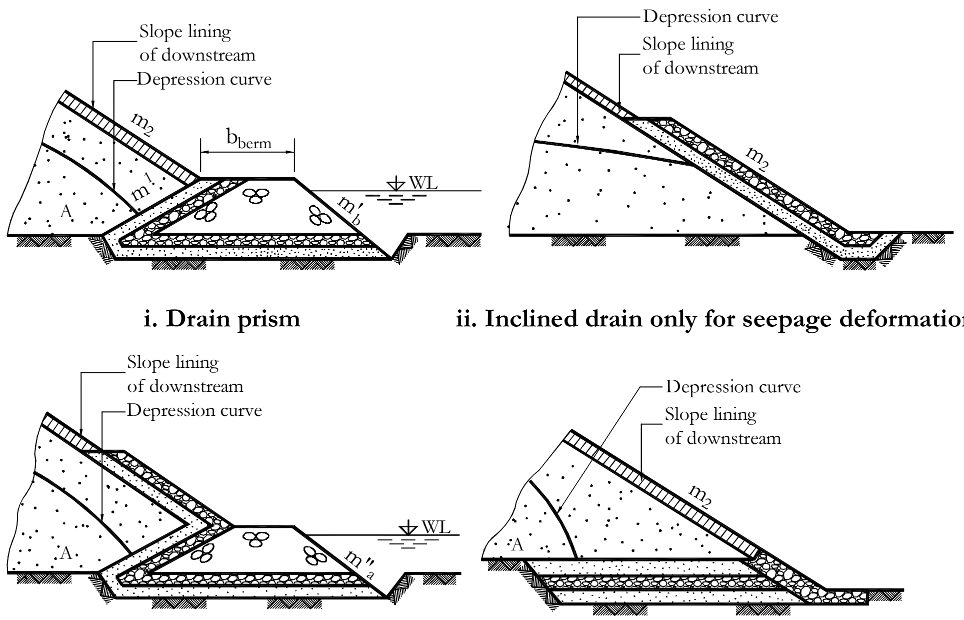
protected with gravel or pebbles mixed soils having a thickness of 10 to 20 cm, from the high velocity of wind. The downstream slopes, if subjected to surface erosion with rainwater, shall also be strengthened similar to the upstream slopes.

Investigation on the flattened slopes of the embankment dams has shown that expenditure on strengthening of the upstream slopes, especially with stones and R.C.C. plates, became a significant part of whole cost of dam. Therefore, the strengthening of slopes can be avoided and they will be stable, if making them with flattened slope. Such slopes resist wind wave action for wave height up to 1.5 m and after some time the slopes with little weathering become in stable condition for further wave action.

For preliminary design of the downstream flattened slopes the diagram made on the basis of natural observations can be easily used Figure 8. The relationships of wave height and slope coefficient for different soils are given on this figure. Taking some factor of safety (1.2 to 1.4) the slope coefficient of unstrengthened slope can be adopted from this diagram.

12.3 Downstream Drains

Drains of the embankment dams serve for lowering the phreatic curve in their body, avoiding the exit of seepage flow to the downstream slope and carrying the seepage flow from the body of dam in the downstream. Different types of drains are shown in Figure 9.



i. Drain prism **ii. Inclined drain only for seepage deformation**

iii. Combined drain prism for short time rise of water level above drain berm **iv. Horizontal drain in absence of water in downstream**

Figure 9: Drains of embankment dams (Nayak, 1993)

For the selection of drain structure it is essential to take into account the type of dams, engineering geological conditions of the foundation and banks, climatic conditions of the construction area, condition of their exploitation, availability of drain materials, location of the drains: in the river bed or floodplain part of the dams and so on.

Intake part of the drains is of filters (also called inverted drains), which are made from non-cohesive natural soils, soil mixture or from fiber synthetic materials. Soil of the inverted filters must be coldproof, not be dissolved by the action of seepage flow.

Drain Prism is considered to be spread type of drain that requires a large volume of stones (Figure 9-i) and functions with the variable level of water in the downstream.

Inclined drain prism shall be composed of a layers of the inverted filters placed on the downstream bottom slope. Strictly speaking, it is not a drain in general understanding because this does not lower the phreatic curve but serves only for the precaution of filtration deformation on the slope (Figure 9-ii)

Combined drain prism is a composition of the drain prism with inclined drain prism (Figure 9-iii). It shall be used for short time rise in the downstream water level above the drain prism.

Horizontal drain prism (Figure 9-iv) consists of a series of layers of the inverted filter capable of draining the dam bed and its foundation. In absence of water in the downstream it shall be used and located above the downstream water level.

12.3.1 Location of Drains

One of the drain problems is to lower the phreatic curve in its same location so that it could prevail at all points from the downstream bottom slope plane at a distance not less than the freezing depth. In this way, the function of drain during winter season is determined by the location of its end point A (Figure 9) embedded into the body of dam. Transfer of this point A to the side of the upstream bottom slope leads to rise in gradients with the entrance to drain, increase in the seepage flow discharges and to damp especially of its intake part during drain maintenance, in case of its siltation.

12.3.2 Inverted Filters of Drains

In the practice of drain design with the inverted filters, different percentage of the filter materials have been used, e.g. somewhere d_{50}/d_{10} or somewhere D_{60}/D_{10}, but it does not affect to the work performance of drains. However, in the approaching zone of seepage flow to the drain, gradients of head rise may cause occurrence of the seepage deformation of soil of the dam body and foundation. To avoid such deformation drains are protected by the inverted filters, partial compositions of which are to be selected with the use of diagrams.

In these diagrams notation d belongs to the soil depth to be protected, D^{I}- to first layer of the filter, D^{II}- to second layer of the filter and so on. With the help of these diagrams there can be made a possibility of use of the natural soil mixtures or to provide a granulometrical curve, parameters of which exist in the ranges of permissible characteristics.

Application of these diagrams can be illustrated with following example. Drain prism of the embankment dam with d_{50}=0.1 mm on the clay foundation is protected by the inverted filters, for which the quarry soil is suggested to use with characteristics: for first layer D^{I}_{10}=0.15 mm; D^{I}_{50}=20 mm; D^{II}_{60}=24 mm. Show applicability of the stated soils for use in the inverted filter. The diverging seepage flow will take place at the contact of dam body with drains. By the diagram (Figure 10-i) for first layer of filter a point having co-ordinates $\eta=D^{I}_{60}/D^{I}_{10}=1.3/0.15=8.6$ and $\xi=D^{I}_{50}/d_{50}=1.1/0.1=11$ falls in the range of permissible characteristics. Thus, this soil can be used for first layer of filter.

For second layer of filter a point with characteristics $\eta=D^{II}_{60}/D^{II}_{10}=24/6=4$ and $\xi=D^{II}_{50}/D^{I}_{50}=20/1.1=18.2$ falls in the range of not permissible characteristics and, thus, quarry soil will not be applicable for filters.

At the contact of drains with clay foundation there is a possibility of deformation in the form of contact low-off vent, evaluation of such type of seepage deformation can be done with the help of Figure 10-v. For first layer of filter with the use of soil having characteristics: D^{I}_{10}=2 mm, D^{I}_{50}=12 mm, D^{I}_{60}=15 mm, for which $\eta=D^{I}_{60}/D^{I}_{10}=15/2=7.5$ the point with parameters η and D^{I}_{50} falls in the range of permissible characteristics and, thus, this soil can be used for first layer of filter. Its second layer on foundation is selected in the same way as second layer on contact with the dam body, but diagram presented in -ii is applied, because here will be rising seepage flow.

Similar calculations are done for selection of diameters of the next layers of inverted filters. The thickness of layers for dry placing is taken to be not less than 0.2 m and 0.5 m for placing in water.

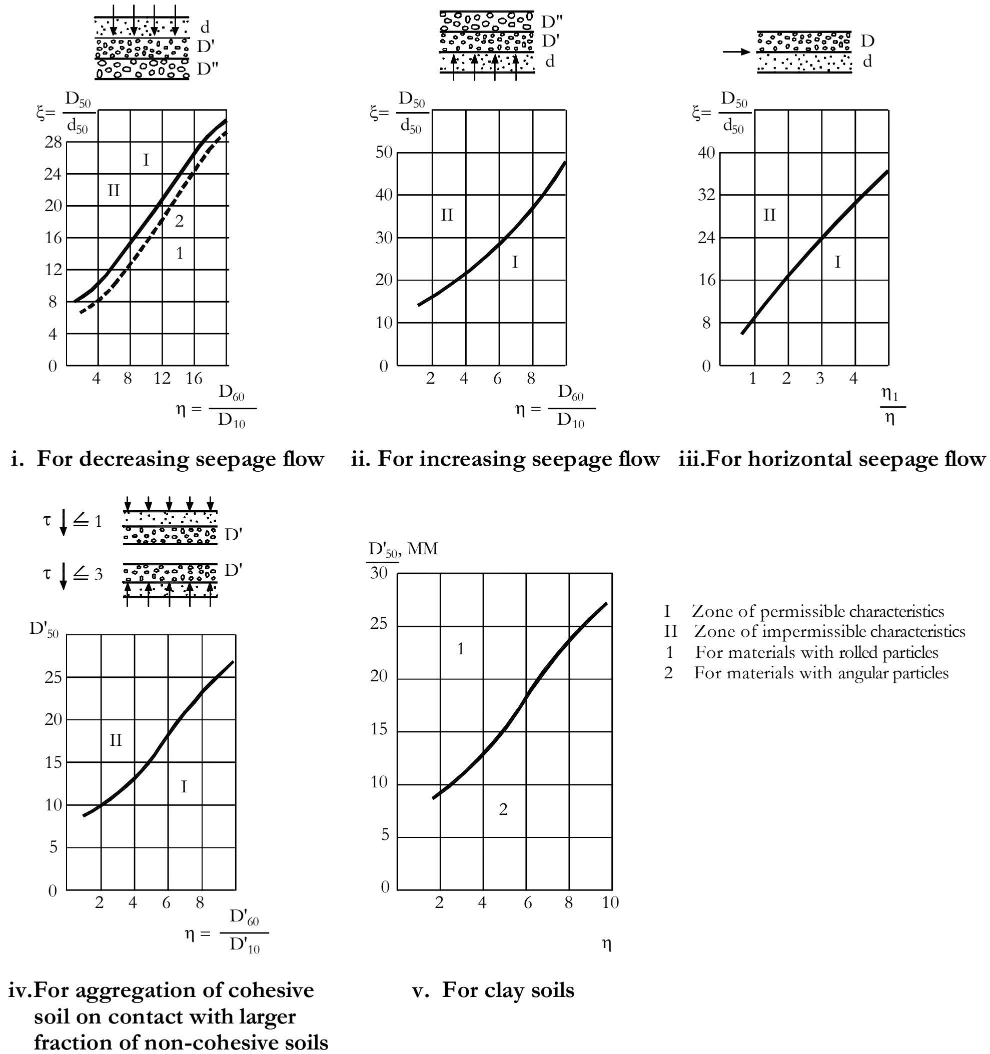


Figure 10: Diagrams for selection and verification of inverted filters in drains of embankment dams (Nayak, 1993)

For the inverted filters of drains the artificial mineral fiber materials may also be used, which are made from the glass or basalt fibers. Such materials have different types and are produced in the form of fiber roll, mats, plates and glass fibres. These materials shall also be used instead of the filter preparation under the concrete and R.C.C. covers of the upstream embankment slopes.

12.4 Anti-seepage Arrangements

Dimensions of the anti-seepage arrangements shall be determined by design and then corrected as per conditions of construction works for performing them by the mechanized method, Generally speaking, thickness of vertical or steep slope of the anti-seepage elements for the embankment dams, i.e. corewalls, screen sheets, water locks, cutoffs and others, by the construction conditions must not be less than 1 to 2 m, and of the horizontal or moderate slope elements, e.g. upstream floor, must not be less than 0.50 m.

Thickness of the soil anti-seepage elements shall be determined by the permissible gradient of head, and in some cases by limit of seepage flow discharges. For corewalls and screen sheets the permissible gradient of head on the basis of construction experience shall be adopted: I_{perm}=8 to 12 (greater value for clay and lower value for silty clay). The value of permissible gradients for the clayey upstream floor may be reduced to 4 to 6. Thickness of the anti-seepage element will be determined by the relationship:

Eq. 10
$$\delta \geq \frac{\Delta H}{I_{perm}}$$

where ΔH is the differential head before and after the anti-seepage element and I_{perm} is the permissible gradient of seepage through the anti-seepage element.

13. SEEPAGE FLOW DESIGN

13.1 Seepage Design of Dams on Impermeable Foundation

In recent years the widely used method of equivalent profile with simple calculations has been recommended for general use. In this method an adopted design sketch of the dam is replaced by the equivalent one in relation to seepage with vertical upstream slope, which (Figure 11) lies at the distance ΔL from the vertical plane (line) drawn through the point of interaction of the water level with embankment slope. The magnitude ΔL is determined by the relationship (Zhurablov, 1975):

Eq. 11
$$\Delta L = \beta H_1$$

where

Eq. 12
$$\beta = \frac{m_1}{2m_1 + 1}$$

Starting from the vertical plane, the phreatic curve is made and its part in approach to the upstream slope has to be corrected visually such that it could be perpendicular to the embankment slope and further be changed into the phreatic curve.

13.1.1 Homogenous Earthen Dam without Drain

Height of the seepage flow existing at the down stream slope is known as the leaking zone(Figure 11-i) and its magnitude can be determined by the formula (Zhurablov, 1975):

Eq. 13
$$h_1 = \frac{L_d}{m_2} - \sqrt{\left(\frac{L_d}{m_2}\right)^2 - H_1^2}$$

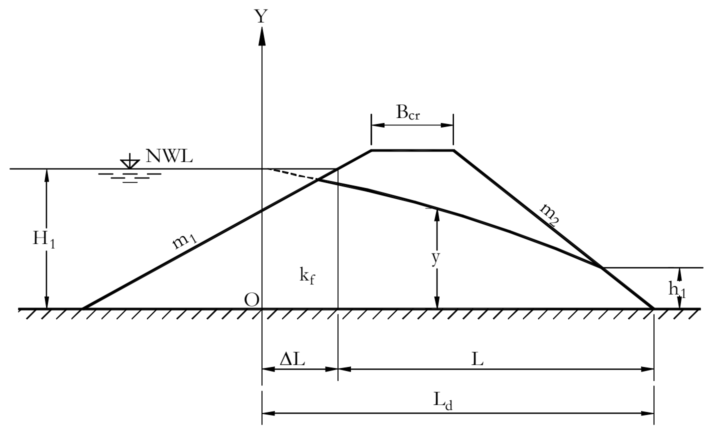
Specific seepage discharge is found by the formula:

Eq. 14
$$q = k_f \frac{h_1}{m_2}$$

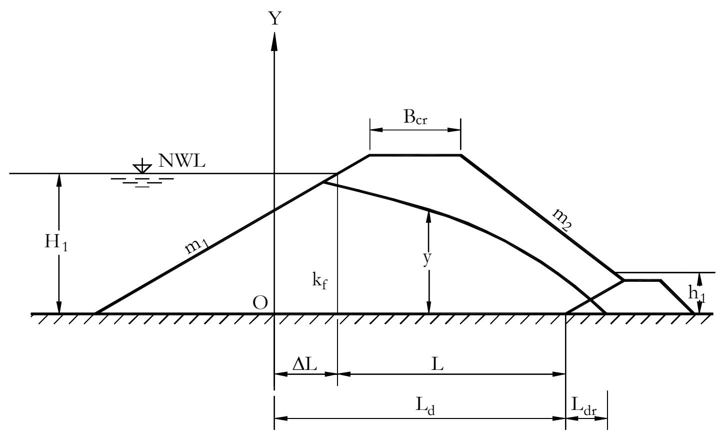
Taking initial coordinate at point O, the phreatic curve is drawn by Dupy's equation:

Eq. 15
$$y^2 = H_1^2 - 2\frac{q}{k_f}x$$

Giving the values of x from zero to $x = L_d - m_2h_1$, the phreatic curve is to be constructed by the help of the formula (Eq. 13) and it has to be corrected by hand at the zone approaching to the upstream slope, as above explained.



i. Homogenous dam without drain prism



ii. Homogenous dam with drain prism

Figure 11: Sketches of seepage calculation (Zhurablov, 1975)

13.1.2 Homogenous Earthen Dam with Drain

In this case (Figure 11-ii) the seepage equation will be:

Eq. 16
$$q = \frac{k_f H_1^2}{2(L_d + l_{dr})}$$

where $L_d = L + \Delta L$ is design seepage length, *m* and l_{dr} is drain length in m.

If in the formula (Eq. 14) length of the drain l_{dr} is to be neglected because of its very small value in the comparison with L_d, it will turn into:

Eq. 17
$$q = \frac{k_f H_1^2}{2L_d}$$

Ordinate of the phreatic curve at the beginning of drain becomes

Eq. 18
$$h_1 = \frac{q}{k_f}$$

From the initial co-ordinate at point O, phreatic curve has to be drawn by Eq. 13.

For x=0, ordinate will be H_1 and for $x = L_d$ it equals to be h_1. For $x=L_d+l_{dr}$ ordinate will be zero and distance from the starting of the drain upto this point is determined by the relationship (Zhurablov, 1975):

Eq. 19
$$l_{dr} = 0.5\frac{q}{k_f}$$

13.1.3 Earthen Dam with Central Core Wall

For the seepage calculation of such dams the method of virtual length is usually applied, for which a central core having mean thickness δ_m and permeability coefficient k_c is replaced by the prism with permeability coefficient k_f. Virtual length of the central core is to be determined by the expression:

Eq. 20
$$L_c = \delta_m \frac{k_f}{k_c}$$

After such replacement, calculation is done in the same way as for homogenous earthen dam without drain or with drain depending upon the achieved calculation sketches. Phreatic curve has to be drawn only in the zone of dam before and after the central core.

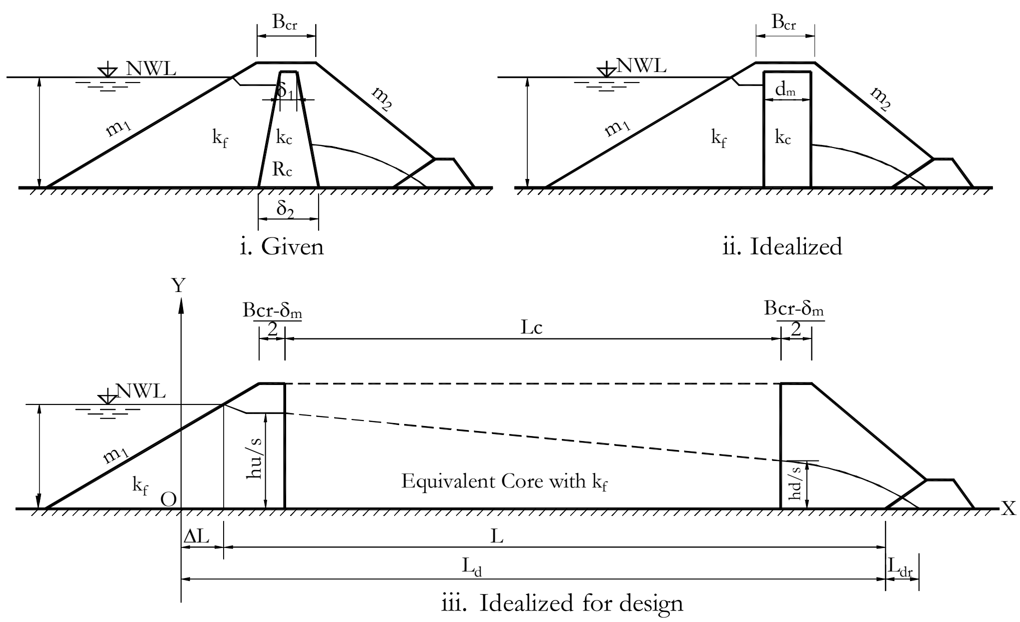


Figure 12: Sketches for seepage calculation with central core (Nayak, 1993)

13.1.4 Earthen Dam with Inclined Core

Seepage calculation of such dams Figure 13 can be done by the different methods. One of them is the method of virtual length which is based on the replacement of inclined core having mean thickness δ_m and permeability coefficient k_c of equivalent prism with permeability coefficient k_f and its horizontal length:

Eq. 21
$$L'_c = \delta_m \frac{k_f}{k_c}\sin\theta$$

where θ is the angle formed between inclined core and horizontal line of the foundation in degree.

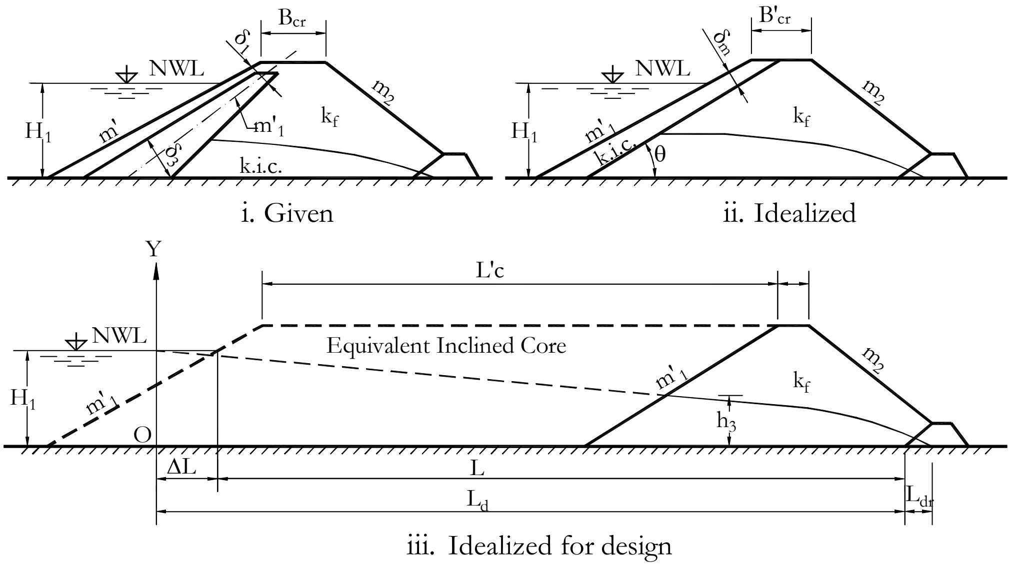


Figure 13: Sketches for seepage calculation of dam with inclined core (Nayak, 1993)

Thus, to the adopted sketch solution is applied for homogenous earthen dam with drain or without drain depending upon the provided structure of the dam. Losses of head in the range of loaded layer of the inclined core are neglected.

13.2 Seepage Design of Dams on Permeable Foundations

13.2.1 Earthen Dam with Screen, Apron and Drain

In the calculation of these dams Figure 14 the losses of head in the protected layer are not considered. Screen and apron are taken into account to be impervious. Slope of the screen is to be adopted along its mean line and fall of head along the length of apron is taken to be straight (linear).

Seepage equation for the different values of permeability coefficient of the dam body k_f and foundation k_0 takes the form (Zhurablov, 1975):

Eq. 22

$$k_0 T \frac{H_1 - h_3}{n\left(L_a + m'_1 h_3\right)} = \frac{h_3}{L}\left(k_0 T + k_f \frac{h_3}{2}\right)$$

Geometrical dimensions entering in the Eq. 22 are shown in Figure 14.

n is coefficient considering the length of seepage flow due to curvature: *n*=1.15 for *L*/*T*=20; 1.18 for 5; 1.23 for 3; 1.44 for 2 and 1.87 for 1.

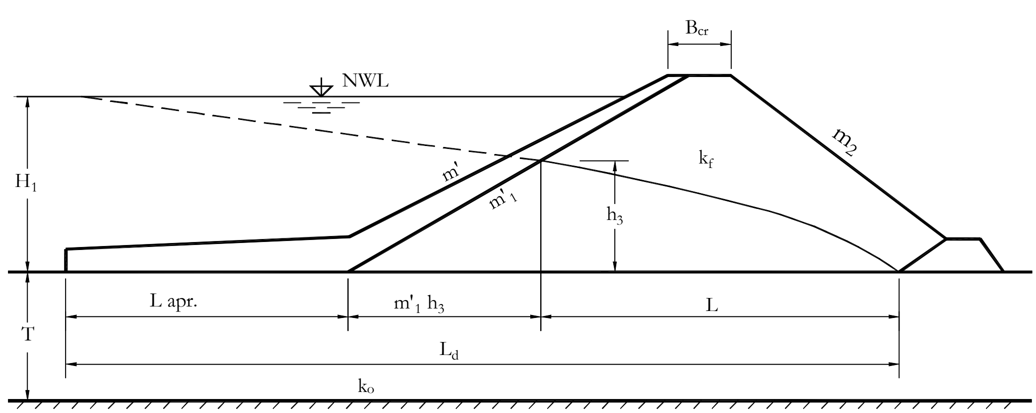


Figure 14: Sketches for seepage calculation of dam with screen and apron (Zhurablov, 1975)

Such equation is easily to be solved graphically, for which the curves for L.H.S. and R.H.S. parts of the equation are drawn for the different values of h_3. The point of intersection of the curves gives actual root of this equation.

Depression curve in the body of dam with drain is to be drawn by the use of Eq. 13, replacing H_1 by h_3 in it.

13.2.2 Homogenous Dams without Drain

Here is possibility that permeability coefficient of the dam body k_f and its foundation k_0 may be the same or different. In the approximate method, the design of seepage flow shall be conducted with two independent assumptions. Initially, it is considered that the dam body is permeable and its foundation – impermeable, and for this case specific seepage discharge q_1 shall be found with construction of the phreatic curve in it. Then other case will be considered, in which the dam body shall be taken into account the impermeable and its foundation –permeable, and specific seepage discharge q_2 will be determined by the formula (Zhurablov, 1975):

Eq. 23
$$q_2 = k_0 T \frac{H_1 - h_1}{nL}$$

where T is the depth of the permeable foundation, L is the width of the dam, n is the numerical coefficient considering the length of seepage flow due to curvature to be adopted depending on the ratio L/T (see Eq. 20).

The total seepage flow discharge shall be obtained as the sum of discharges through the dam body and its foundation:

Eq. 24
$$q = q_1 + q_2$$

It is essential to keep it in mind that in this approximate method, location of the phreatic curve shall be little bit higher and more higher with enough difference in the permeability coefficient of the dam foundation and its body for the design case with $k_0 < k_f$.

14. STABILITY OF EMBANKMENT SLOPES

14.1 Determination of Safety Factor

Problems on the slope stability of the earthen dams are that in some conditions slope of those looses stability, sliding of a part of the embankment massive takes place, catching partially the foundation soil.

Knowing the physico-mechanical properties of soil of the dam body a foundation, design on the slope stability provides a minimum value of the safety factor, which must be equal to or more than the permissible value. The class of structures determines its value Table 5.

Table 5: Permissible safety factor of thrust stability, ξ_{sperm}

Loading condition	Permissible safety factor of thrust stability for class of hazard			
General	1.30	1.20	1.15	1.10
Special	1.10	1.10	1.05	1.05

(Source: Zhurablov, 1975)

In literature there are various methods for the determination of safety factor of the embankment slope stability. The effective one of them is the slip circle method. Obtained safety factor for general loading condition must not exceed by 15% and for special loading condition by 30% with respect to permissible safety factor.

Slope stability of the embankment is evaluated by the factor of safety which is the ratio of moments created by the supporting forces to moments created by the shearing forces with respect to some point O, placed out of the dam contour, above its crest. In general case, the design formula takes the form (Nayak, 1993):

Eq. 25
$$\xi_s = \frac{\Sigma M_{\sup}}{\Sigma M_{th}}$$

Calculation on the slope stability is followed to be done for one meter length of the dam. For determination of the factor of safety all the acting forces are essentially to be transferred on the slip surface, except seepage (hydrodynamic) force which is considered as volumetric force. If the characteristics of soils of the dam body and foundation are not same, slip prism is vertically divided into a series of columns having width b = 0.1R (that simplifies computations of the trigonometric functions in the formula) except last, width of which gets less. Numbering is done from the zero column, placed symmetrically with respect to vertical which is drawn from the centre of rotation Figure 15. When the soils of vertical columns have different volume mass, its height for the computation is taken to be mean weighted height with constant volume mass. For averaging this height, the formula will be (Nayak, 1993):

Eq. 26
$$h_m = h_1 \frac{\gamma_1}{\gamma_m} + h_2 \frac{\gamma_2}{\gamma_m} + ... + h_n \frac{\gamma_n}{\gamma_m}$$

where h_1, h_2, ……h_n are the separated height of the soil mass in columns, in which γ_1, γ_2, …….γ_m are the volume mass respectively and γ_m is mean volume mass.

Usually, mean height is determined for soils of the dam body with natural humidity but it is possible to adopt other soil.

Safety factor of the soil mass separated by the slip curve has to be determined by the formula (Eq. 25) with summing stability of the different vertical columns (Nayak, 1993):

Eq. 27
$$\xi_s = \frac{b\Sigma h_m \gamma_m \cos\alpha \tan\varphi + \Sigma c \cdot l}{b\Sigma h_m \gamma_m \sin\alpha + \phi \frac{r}{R}}$$

where h_m is the mean weighed height of the vertical columns, to be taken from the drawing in scale, α is the variable angle (Figure 15), φ is the angle of internal friction of soil along the slip surface; c is the specific bond force along the slip surface, ϕ is the hydrodynamic (seepage) force; R is the radius of the slip curve and e is the arm upto the centre of gravity of the resultant (from the seepage forces) to be taken from the drawing in scale.

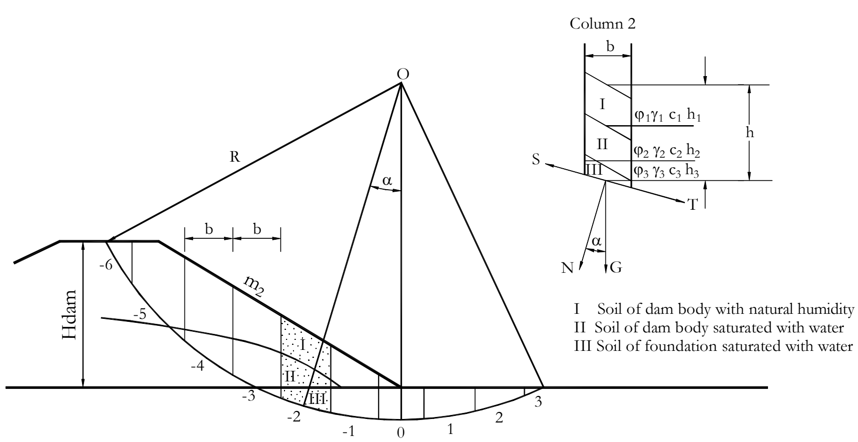


Figure 15: Sketch for determination of factor of safety for slope stability with slip circle method (Nayak, 1993)

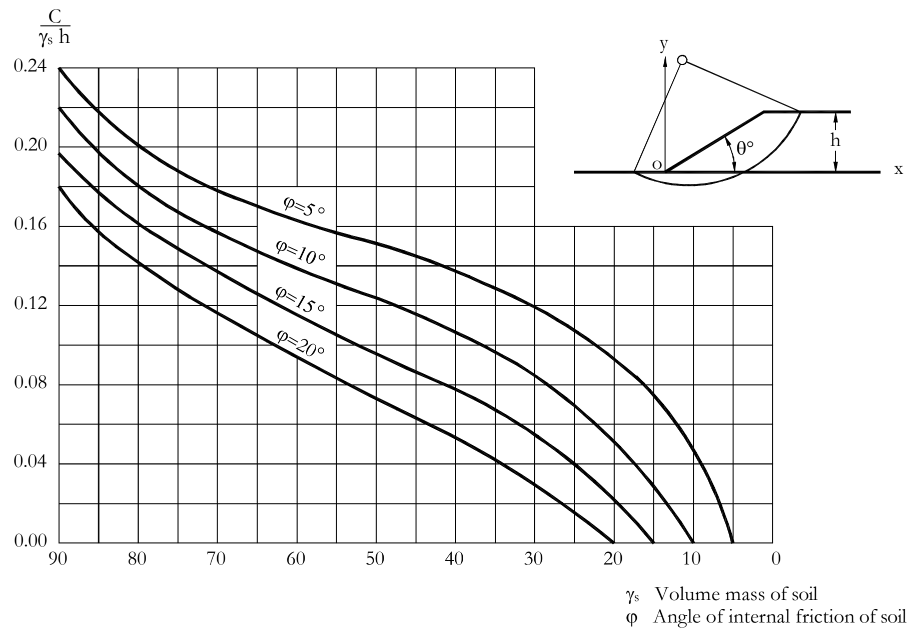


Figure 16: Diagram for design of slope stability of homogenous soils (Zhurablov, 1975)

14.1.1 Consideration of Seepage Force

In the body of dam and its foundation the seepage force occurs with presence of seepage flow, which can be taken into account as the volumetric force applied to the slip circle. It shall be determined by the formula (Zhurablov, 1975):

Eq. 28 $$\phi = \gamma_w V I_w$$

where, γ_w is the unit weight of water, V is the volume of soil mass and I_w is the gradient of head in the zone considered. Presence of this force decreases the slope stability.

15. DESIGN OF SETTLEMENT OF DAM BODY AND FOUNDATION

15.1 Stabilized Settlement

From the moment of applying external force to the compressible foundation, settlements may be extended up to a long period, sometimes years. The final settlements in the embankment dams are known as a stabilized settlement.

Delay of settlements in time after the application of forces will be in soils saturated with water having an inadequate value of permeability coefficient. The final stabilized settlements in the clayey soils will take place for a long time, while in the sandy and gravel mixed soils it rapidly ends during the construction period of structures.

The design of stabilized settlements shall be performed with the verticles, number of which in the cross-sectional profile of dam is adopted not less than three. The design formula is (Zhurablov, 1975):

Eq. 29 $$\Delta h_s = h_1 \frac{\varepsilon_1 - \varepsilon_2}{1 + \varepsilon_1}$$

where Δh_s is the stabilized (final) settlement, h_1 is the depth of the compressible layer (active zone) of soil before applying external forces, ε_1 is the porosity coefficient of soil before applying force and ε_2 is the porosity coefficient of soil after applying force.

The values of coefficients ε_1 and ε_2 shall be found with the help of compressive curves of soils. The design by the formula (Eq. 29) is performed layer by layer, for which the thickness of layers must correspond to the character and thickness of the foundation soil zone and is adopted not more than 10% of thickness of the active zone. The porosity coefficients ε_1 and ε_2 are needed to transfer to the middle of each layer.

Eq. 29 holds good for the condition of plane problem, i.e. for the continuous uniformly distributed load, placed on the unlimited area. Generally speaking, the force (load) of the embankment dam satisfies to this condition.

15.2 Unstabilized Settlement

If the settlement of the embankment dam is determined at the end of the given period of time, when it fully not compacted, this settlement is known as unstabilized. Approximately, it shall be determined by the empirical formula (Zhurablov, 1975):

Eq. 30

$$\Delta h_t = \Delta h_c \left(1 - 2.7^{-0.5t}\right)$$

where Δh_t is the settlement for time t in years from the start of load application and Δh_c is stabilized settlement calculated by the Eq. 29.

The Eq. 30 shall hold good for the ratio of thickness of the compressible layer to the width of dam at the foundation will be more than unity. For less values of this ratio the exponential power of the equation will be more.

16. ROCKFILL DAMS

In the areas of adequate availability of the rocks/stones, rockfill dams are used for creation of the reservoirs or pressure head at the headworks. In this case, the stone placed in the form of trapezoidal prism serves for the mass weight that resists the shear forces coming from the upstream hydro-static pressure of water.

16.1 Cross-sectional Profiles

The rock-fill dams are a composition of the rocks/stones-filled and soil. The rock-fill part in such dams provides/creates pressure head in the reservoir and earthen part serves for anti-seepage arrangement essential for reducing losses of water against seepage concerning to advantages of this dam, the time for construction of the earthen anti-seepage arrangements shall be limited during (cold) negative temperature and rainy seasons. Besides, the affect of rock/stone settlement seems less on the function of anti-seepage arrangements and the strengthening (pitching) of upstream slope, especially if it is made of stones.

16.2 Principal Characteristics

The body of dam composed with stone filters water. For decrease of loss of water on seepage in the rockfill dams, impervious walls are placed on the upstream slope or foundation in different pose. Rockfill dams are built up practically on any foundations: rocky and non-rocky. These dams do not need special arrangement of the drains, in which there is absence of back pressure affecting the stability. All types of rockfill dams are seismically stable and their height for construction has not been limited. Its height can be raised without breaking the work of headworks.

16.3 Types of Rockfill Dams

According to the location of anti-seepage arrangements in the cross-section profile, rock-fill dams are divided into the following types (Figure 17):

a. Dams with earthen screen;

b. Dams with central earthen core;

c. Dams with upstream earthen prism and with central earthen prism.

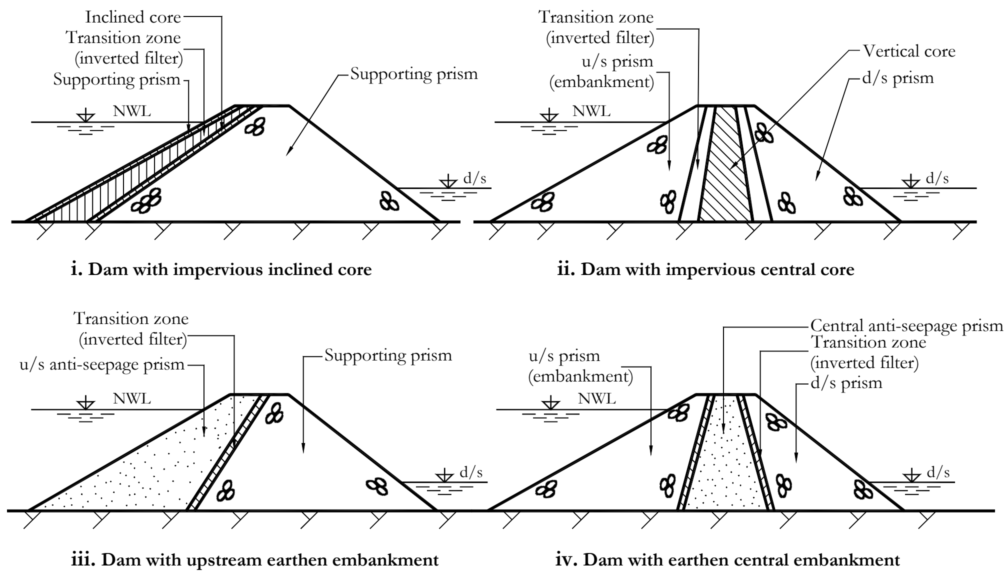


Figure 17: Types of rockfill dams (Nayak, 1993)

The dams having the upstream and central earthen prism, by principle, are a diversification of dams with earthen screen or core. In these dams, the dimensions of earthen element are adequately increased that is needed from the condition of seepage safety. In recent years the rock-fill dams are widely used in the world, where synthetic materials are playing a main role for anti-seepage arrangements. Clay, silty clay, and rarely a mixture type of clay-concrete are used for making anti-seepage earthen materials.

Application of the polymer films shall be taken in to account for their full impermeability, elasticity and withstand against corrosion. They are placed in the zone of upstream slope of dams of screen type protected from both the sides by the soil layers. At the river bank slopes and foundation the polymer films shall be placed about the concrete cut-offs.

16.4 Anti-Seepage Arrangements

16.4.1 Dimensions of Anti-seepage Arrangements

The earthen core wall and screen possess the variable thickness. In the upper part their dimensions shall be determined by the conditions of construction works, and any horizontal section including foundation, by the design. The thickness of core wall and screen are usually taken not less than 0.15 of head over the dam. Such dimension avoids seepage deformation in the anti-seepage arrangements.

The following requirements for the solids of screen and core of the rock-fill dams are essential: weak permeability avoiding enough seepage losses of water through the dam body; plasticity permitting deformation of soil without creak formation; resistance to shear as well as good compressibility of soil. The natural clayey silty soils, and silty sandy as well as moren depositions practically satisfy these conditions. The characteristic of the permeability coefficient of order of 10^{-6} to 10^{-7} cm/sec and angle of internal friction of 20^0 to26^0.

16.4.2 Transition Zones

The transition zones shall be placed at the joint (boundary) of the earthen anti-seepage arrangements rockfills. They are made in the form of several soil layers with the different particle sizes in each layer. In approaching core and screen, soils having more fine particles will be placed increasing them to the side of rockfill prisms. Diameter of particles of the transition layer shall be selected by the type of inverted filters. Such selection does not allow the penetration of fine particles through the pores of bigger contactable layers, and avoids the seepage deformations in the presence of seepage flow.

The thickness of layers shall be adopted by the condition of workability of the construction works, considering that whole thickness must not be less than 3.0 to 3.5 m. The thickness of layers in the upper part of the dam is allowed to decrease little-bit, where a difficulty occurs for their allocation (layout) with the small crest width. Material for the transition zones serves for non-cohesive, permeable sandy, sandy pebble and pebble gravel mixed soils, and coarse aggregate as well as disposal of the stone crusher plants.

16.4.3 Transition of Anti-seepage Arrangements with Foundation

In the rock-fill dams, the foundation and their slopes posses the creaked rocks, there are arranged grout curtains, they shall be placed at the middle of core-wall or screen in order to obtain continuous less-permeable line on the whole length grout curtain, core and screen.

The transition of grout curtain with the help of concrete cut-off embedded in the foundation. In the rocky foundations having a less degree of water is small against seepage, the core wall and screen (cover) shall be transited through the concrete walls without the grout curtain. In the foundations composed from the non-rocky soils, the deep cut-off walls are made instead of the sheet piles are driven, locating them also at the middle of cut-off or core wall

16.4.4 Porewater Pressure

The porewater pressure occurs in the less permeable soils saturated with water due to compaction under in the action of external load and self weight of the soil mass. As the flow of the water discharge from soil, load starts to transfer to the soil particles and, when water from the pores will be extracted, pressure fully transfers to the soil particles, which is known as effective pressure. Thus, the sum of pressure to be taken by the pore water and soil particles will be equal to the external pressures. The character of distribution of these two pressures in time may be observed on Figure 18, from which there is followed that with decrease in the permeability coefficient of soil. The pore water pressure is observed for a long time.

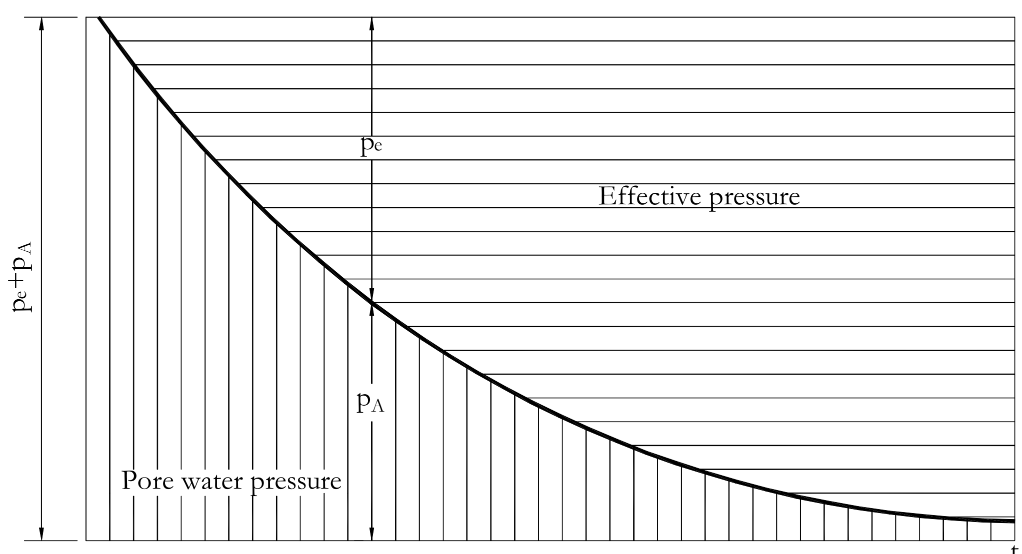


Figure 18: Variation of the pore water pressure (neutral pressure) in time (Zhurablov, 1975)

If the shear of soil is possible in which being pore water pressure, the capacity the soil to take shear forces will be inadequate because the normal force transfers to water and, thus, there is no reactive force, i.e. friction force. For decreasing pore water pressure it is necessary as quick as possible to extract water from the soil mass and there occurs a friction force.

17. SERVICE LIFE OF RESERVOIR

The degree of sediment concentration in the river flow mainly determines the service life of the reservoir, on which depends the economic effectiveness of the hydropower project to the constructed in the country.

For evaluation of life of the reservoir, there exists various methods and formulae in literature, which do not consider the bed sediment concentration, reservoir bank erosion and sediment flushing downstream and provide not reliable results. Considering this concentration an effective formula developed by Prof. J. N. Nayak (1993) for determining the life of a reservoir due to its sedimentation, can be applied:

Eq. 31

$$T = \frac{V}{\frac{\zeta_{tr} V_0}{\gamma_1}\left(1 + \frac{\gamma_1}{\gamma_2}\right) + V_b - V_{exit}}$$

where ζ_{tr} is the suspended sediment transporting capacity of one cubic meter of the flow, V_o is the mean annual flow of the river, γ_1 and γ_2 are the volume weights of suspended and bed sediments, respectively, V_b is the long term mean annual volume of sediments deposited in the reservoir due to erosion of its banks and V_{exit} is the long term mean annual volume of sediments outleting through the flushing tunnels in the similar river basins.

18. EVALUATION OF EARTHQUAKE

18.1 Seismic Effect

Evaluation of seismic effect for embankments located area of low seismicity may be accomplished using the seismic coefficient in the pseudostatic method of analysis. This investigation need only be applied to those critical failure surfaces found in analyzing loading conditions with out earthquake loading.

For embankments located in area of strong seismicity, a dynamic analysis of embankment stability should be performed based on present state of-the-art procedures for the earthquake loading to be used in dynamic analysis.

In general, an embankment dam should be capable of retaining the reservoir under condition induced by the maximum credible earthquake where failure would causes loss of life. The following investigations should be accomplished for all proposed embankments, with the exception that existing conformed "low" hazard potential dams may be exempted from these investigations:

a. A seismic stability investigation using a dynamic analysis for proposed and existing dams located in seismic zones;

b. An elevation of the liquefaction potential for all dams that have or will have liquefiable materials either in the embankment dam or foundation;

c. A geological and seismological review of existing dams in seismic zones to located faults ascertain the seismic history the of region around the dam and reservoir; and

18.2 Factors of Safety

The factor of safety includes a margin of safety to guard against ultimate failure, to avoid unacceptable deformations, and to cover uncertainties associated with the measurement of soil properties or the analysis used. In selecting a minimum acceptable factor of safety an evaluation should be made on both the degree of conservation with which assumptions were made in choosing soil strength parameters and pore water pressure, and the influence of the made of analysis which is used.

A qualitative estimate of the factor of safety can be obtained by examining conditions of equilibrium when incipient failure is postulated, and comparing the strength necessary to maintain limiting equilibrium with the available strength of the soil.

Therefore, the slope stability analysis of soils requires measurements of the shear strength and computation of the shear stress. Appropriate minimum values of factors of safety to be used in the stability a slope depends primarily on the measurement of strength. Factors including the selection of minimum factors of safety include:

a. Reliability of laboratory shear strength testing results;
b. Embankment height;
c. Storage capacity;
d. Thoroughness of investigations;
e. Construction quality, construction control of embankment fills;
f. Judgment based on past experience;
g. Design conditions being analyzed.
h. Predictions of pore water pressure used in effective stress analyses.

Final accepted factors of safety may depend upon the degree of confidence in the engineering data available. In the final analysis, the consequence of a failure with respect to human life, property damage and importance of project function are important consideration in establishing factors of safety for specification. The minimum factors of safety are given in Table 6.

Table 6: Minimum factors of safety

Loading condition	Minimum factor of safety	Slope to be analyzed
End of construction condition	1.3	Upstream and downstream
Sudden drawdown from maximum pool	> 1.1	Upstream
Sudden drawdown from spillway crest or top of gates	1.2	Upstream
Steady seepage with maximum storage pool	1.5	Upstream and downstream
Steady seepage with surcharge pool	1.4	Downstream
Earthquake (for steady seepage condition with seismic loading using the seismic coefficient)	> 1.0	Upstream and downstream

(Source: ASCE, 1989

The degree of safety against ultimate may be determined as:

Eq. 32
$$FS = \frac{S}{T}$$

where, FS is the factor of safety, S is shear strength along the trial shear surface and T is equilibrium shear stress along the same trail shear force.

18.3 Seismic Stability Evaluation

Various methods of analysis are available for evaluating the seismic stability of an earth dam. These may be classified as:

- Pseudostatic method;
- Simplified methods; and
- Dynamic method.

Regardless of the method of analysis, the final evaluation of the seismic safety of the embankment should be based on all pertinent factors involved in the investigation and not solely on the numerical result of the analysis.

18.3.1 Analysis Approach

Analysis for earthquake loading should begin with simplified procedure and proceed to more rigorous methods of analyses as a particular situation may warrant. Projects with well compacted embankments and dense foundation soils located in seismic zone and all confirmed low hazard potential projects, may be evaluated by the pseudostatic method using the seismic coefficient assigned to the seismic zone the project is in.

In area of service and/or frequent seismic loading such as in seismic or where foundation liquefaction potential exists, more rigorous dynamic methods of analyses will be necessary. Site specific seismic evaluations will be performed for all projects not covered in the paragraph above. These studies will identify earth quake source area, the maximum credible earthquake, and the distance from the site of each relevant source area. Potential for fault rupture in the dam foundation and in the reservoir will be assessed. The modes of failure that need to be investigated and the appropriate methodology are described in the following subsections.

18.3.2 Modes of Failure

The analysis shall evaluate the following potential effects of earthquakes on embankments:

- Loss of stability
- Excessive Deformations
- Other Mechanisms
- Overtopping due to seiches
- Movement along a fault passing under the dam
- Landslides in abutments causing direct damage to the dam or due to wave in reservoir (Vaiont dam)

18.3.3 Method of Analyses

18.3.3.1 Pseudostatic Analysis

For many years the standard method of evaluating the safety of embankment dams against sliding during earthquakes has been the pseudostatic method of analysis. In using this approach no special consideration has been to the nature of the slope forming or foundation materials and if the computed factor of safety was longer than unity, it has generally been concluded that the seismic stability question has been satisfactorily resolved. In Terzaghi's opinion, depending on the nature of the slope forming materials, a slope may remain stable if the factor of safety is less than unity or may fail if the factor of safety has been found to be grater than unity based on the pseudostatic approach. This has been confirmed by embankment, more sophisticated analyses should be performed.

In general, therefore, earthquake analyses using the seismic coefficient method may be performed only for structure proposed or existing in Seismic Zone. Seismic coefficient at least as large as should be employed in the analysis. In Zones of low hazard potential in other zones where the pseudostatic method of analysis does not necessarily evaluate appropriate the safety of an embankment, more sophisticated analyses should be performed.

18.3.3.2 Simplified Analysis

Following a detailed study of embankment dam performance during earthquakes, seed observed that the seismic resistance of dams constructed of clayed soils is much higher than that of embankments constructed of saturated sands or other cohesionless soils. Thus for embankments which do not involve saturated cohensionless soils, the pseudostatic method of analysis may still be used; alternatively, methods for evaluating deformations in such dams have been developed. The computed displacements can be compared to allowable displacements to determine the adequacy of the embankment.

18.3.3.3 Analysis for Liquefaction Potential

When embankments and/or their foundations are composed of loose sands, silts, or gravels, the pseudostatic method may not be applicable. Therefore, analyses must be performed to determine (a) if liquefaction potential exists and (b) whether such a liquefied condition can lead to failure or excessive deformations of an embankment. There are various simplified method available for evaluating soil liquefaction potential based on empirical correlations between in situ behavior of sands and standard penetration resistance. In addition, methods exist to assess the liquefaction potential of soil by determining whether the soil is contractive or dilative. Under cyclic loading of sufficient magnitude and duration, loose saturated sand, silt, or gravel having a contractive structure will develop high pore water pressures, lose a large portion of its resistance to deformation, and flow.

18.3.3.4 Analysis for Loss of Stability

The potential for loss of stability can be analyzed using a conventional stability analysis (Eq. 32) incorporating minimum strength values corresponding to the degree to which pore water pressures are generated in the soils by the earthquakes shaking. Where the pore pressure ratio in the soil builds up to a value close to 100%, the soil is considered to have developed a condition of liquefaction.

The determination of those zones where liquefaction or pore pressure build-up will occur must be made using a dynamic analysis to determine the stresses and strains induced in the embankment by the maximum anticipated earthquake motions and a knowledge of the pore pressure generation characteristics of the soils comprising the embankment and its foundation. In general clayey soils do not appear to develop increases in pore pressure due to earthquake shaking. However cohesionless soils are highly vulnerable to pore pressure development depending on their relative density and other characteristics which should be considered in the seismic evaluation.

Once the degree of pore pressure build up has been evaluated, and zones of potential liquefaction identified, soil may be assigned strength values for use in a stability analysis

18.3.3.5 Deformation Analysis

Deformation computations are applicable only to dams not subject to a liquefaction (stability) failure.

Deformations can be assumed not to be a problem if the dam is well-built (densely compacted) and peak accelerations are 0.2g or less. If this condition is not satisfied, a deformation analysis should be made.This analysis can be made using a simplified Newmark procedure. The deformation calculated along the failure plane by this method should not generally exceed 60 cm. Large deformations may be acceptable depending on available freeboard, ability of the embankment to heal cracks and other consideration.

18.3.3.6 Other Methods of Analysis

Other failure mechanisms identified in Section 18.3.2 require special methods of analysis which would need to be adapted or developed for the special circumstances of the project. Generally dams located over faults that could potentially move during an earthquake should not permitted unless filter transition zones are provided which are at least twice the maximum potential fault movement both horizontally and vertically.

PART 2E – FISH PASS STRUCTURES

TABLE OF CONTENTS

Part

2E

Fish Pass Structures

1. PURPOSE

Part 2E of the *Design Guidelines for Headworks of Hydropower Projects* provides technical criteria and procedural guidance for the design of fish pass structures for headworks of run-of-river hydropower projects in Nepal. The guidelines are intended to ensure effective design of these structures, with particular consideration of fish species found in Nepali rivers.

2. SCOPE

The guidelines cover the design of fish pass structures. They discuss different types of fish pass structures and provide guidance on selection of the most suitable fish pass structures, especially for the fish found in Nepali rivers. They also deal with the design requirements and procedures for these structures.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Attraction flow	Water external to fish ladder used to attract fish to the general area of the fish ladder entrance.
Fish bypass system	Structure provided to facilitate the downstream migration of the fish when the dam gates are closed.
Fish ladder	Structure constructed to provide fish with a hydro-biological condition which is optimal for leaping.
Fish way	Waterway specifically designed to afford fish passage around a particular obstruction.

4. DESIGN OBJECTIVE

Fish pass structures shall be designed to ensure safe passage of migratory fish from the upstream of the diversion structure to its downstream and vice versa.

5. SCOPE OF DESIGN

The design objective stated in Section 4 shall be achieved through the following activities:

a. Selection of an appropriate location for fish pass structure.
b. Selection of appropriate type of fish pass.
c. Hydraulic design of fish pass.

6. TYPES OF FISH PASS STRUCTURES

Normally, the following types of fish pass structures may be used to facilitate fish migration at the headworks run-of-river hydropower projects:

a. Fish lock.
b. Fish ladder.
c. Fish bypass system.

6.1 Fish Lock

A fish lock may be used to transfer fish from the upstream to the downstream of the diversion structure, or vice versa, as the case may be. It shall consist of a lock, similar to a navigation lock, into which a group of fish may be admitted and mechanically transferred to its destination.

6.2 Fish Ladder

Fish ladders shall be provided to facilitate migration of fish through their leaping action. The ladders may be of one of the following types:

a. Inclined chute type fish ladder.
b. Pool and weir type fish ladder.
c. Pool and orifice type fish ladder.
d. Pool and jet fish type ladder.
e. Denil fish type ladder.

6.2.1 Inclined Chute Fish Ladder

Inclined chute type fish ladders shall consist of checks or baffles which are arranged to provide a zigzag course to the discharging water by driving it from side to side. These ladders may of the following two types:

a. Paired obstacle fish ladder
b. Alternate obstacle fish ladder.

The paired obstacle fish ladder (Figure 1) shall consist of a straight rectangular channel with a sloping pairs of obstacles or baffles spaced along the channel at intervals of about the width of the channel and a straight free passage between them. Its floor in the free passage shall usually be flat, although small bottom obstacles may be used.

The alternate obstacle fish ladder (Figure 2) shall consist of a straight rectangular channel with baffles placed alternatively along the sides, producing a jet deflection in the horizontal plane. The zigzag path of the flow in this ladder is much longer than the fish pass.

6.2.2 Pool and Weir Type Fish Ladder

The pool and weir type fish ladder (Figure 3) shall consist of a series of pools created with small drops in the water surface at the cross walls, allowing fish to leap from one pool to the other. The drops shall generally be about 0.3 m, but they could be as high as 0.6 m at short ladders where only a few jumps are necessary. Long ladders may be provided with intermediate rest pools.

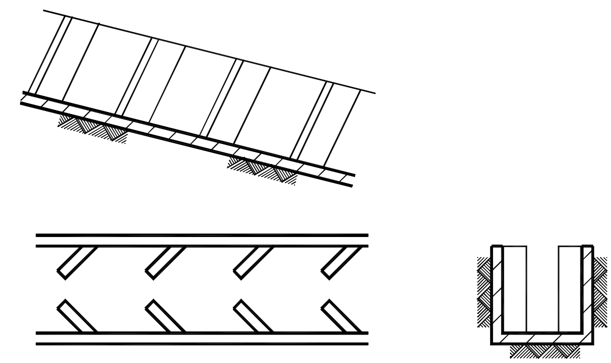


Figure 1: Paired obstacle type fish ladder

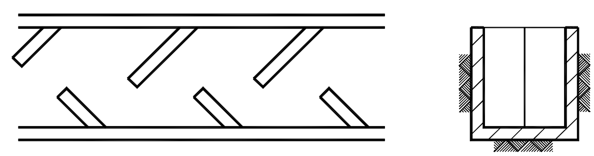


Figure 2: Alternate obstacle type fish pass

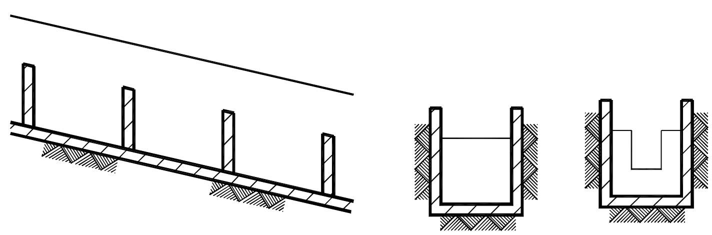


Figure 3: Pool and weir type fish ladder

6.2.3 Pool and Orifice Type Fish Ladder

The pool and orifice type fish ladder shall consist of a submerged orifice through which the entire flow between pools is passed, allowing the fish ascend from pool to pool through the orifices. This ladder may of two types:

a. Coil type (Figure 4).

b. Submerged orifice type (Figure 5).

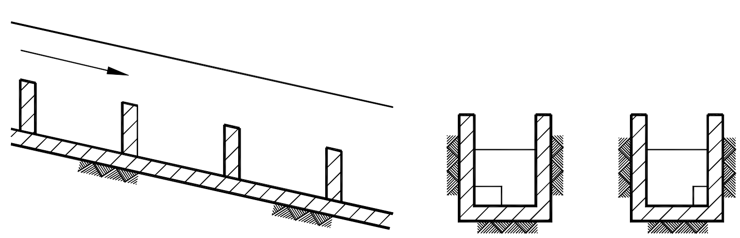


Figure 4: Coil type fish ladder

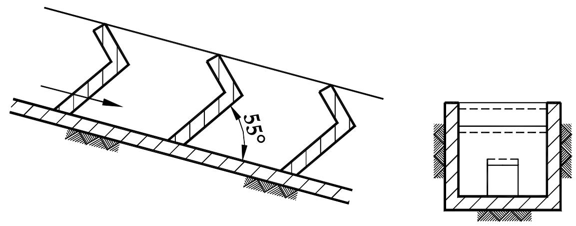


Figure 5: Pool and submerged orifice type fish ladder

6.2.4 Pool and Jet Type Fish Ladder

This ladder shall be similar to the pool and orifice type ladder with the exception that its orifice shall extend over the full height of the baffle in the form of a vertical slot rather than a submerged orifice.

6.2.5 Denil Type Fish Ladder

The Denil or channel type fish ladder shall consist of a straight channels with closely spaced baffles set at an angle with the axis of the channel. The baffles may be formed from oblique and perpendicular parts (Figure 6) or from parallel baffles at the sides placed at an angle with the side walls and the bottom (Figure 7).

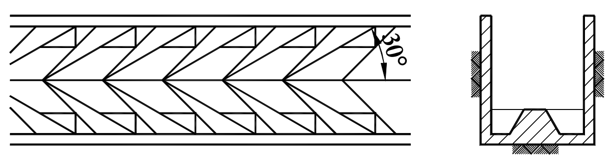


Figure 6: Denil type fish ladder (alternative 1)

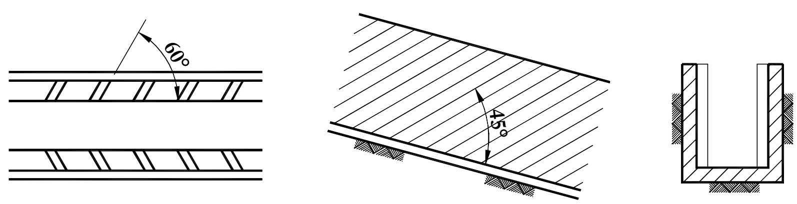


Figure 7: Denil type fish ladder (alternative 2)

6.3 Fish Bypass System

The fish bypass system shall be provided to facilitate the downstream migration of the fish. It shall be provided to facilitate fish migration mostly in the dry period (November to May) when the overflow section gates are closed.

7. SELECTION OF TYPE OF FISH PASS STRUCTURE

The selection of the type of fish pass structure shall depend on the following factors:

a. Suitability or attraction of the structure to the migratory fish.

b. Economy in construction and operation.

Of these, the first factor shall usually govern the selection of the fish pass structure. Model studies may be conducted for selection of the appropriate type of fish pass and to formulate its design.

Since fish diversity is high in Nepal, the choice of the fish pass structure shall ensure that it is suitable to most of the migratory species present in the river. In general, the behavior of the important migratory species such as Snow trout, Golden *Mahseer* and Copper *Mahseer* shall be considered in ladder design. A pool and weir type fish pass may be suitable for most of the fish species found in Nepal.

8. DATA REQUIREMENTS

Planning and design of fish pass structures shall be based on data on the fish species and the river hydraulics. These data shall be collected through thorough investigations and analysis.

8.1 Data on Fish Species

The following data related to the fish species in the river shall be needed for design:

a. Type of fish, their characteristics, habits, instincts and environmental behavior.
b. Place of spawning, whether at particular spots or anywhere.
c. If anadromous in nature, location map of the river/streams showing the migratory route and spawning grounds.
d. Timing of upstream and downstream migration, nature of river flows, turbidity and temperature of water during these periods.
e. Maximum number of fish passing per hour during peak migration period.
f. Requirement of water per fish when confined to keep it in normal condition.
g. Nature of migration, whether in groups or in line.
h. The number of fish passing upstream of the proposed structure for the previous periods, say for a period of 30 years.
i. Commercial value of the fisheries.
j. The normal swimming speed of fish, its darting speed, and preference for swimming or jumping.
k. The stimuli of fish, whether it is attracted by large volume of water or greater velocity.
l. The normal time that fish takes from downstream to spawning grounds and the delay which it is capable of withstanding without any detrimental effect on its biological functions.
m. The size of downstream migrants.
n. The nature of spawning ground and the river conditions at the time of spawning, namely, velocity, depth, turbidity and temperature of water.

8.2 Data on River Hydraulics

The following data related to river hydraulics shall be required for design:

a. River velocity and river discharge.
b. Locations of points of turbulence, upwelling and intensity of surge.
c. Range of discharge likely to be available during the period of migration, along with the water surface profiles for each river bank corresponding to this range.

9. LOCATION

The fish pass shall be provided near the deep channel through which they always move about. Its location shall be readily found by the migrating fish as otherwise the fish may never enter the fishway.

The entrance and exit of the fish pass shall be located away from the range of heavy overfall of water from the diversion structure. The fish pass shall usually be located near the divide wall of the structure between the sluice bays and spillway bays, since water would always be available near the divide wall.

Attraction water may be used to attract fish to the general area of the fish ladder entrance. At gated diversion structures or weirs operated with stop logs, this may be provided simply by adjusting the gates or stop logs so that extra flow is released besides the fish ladder entrance in low flows. Other attraction mechanisms, such as powerful electric light, in the fish ladder entrance area may be adopted.

10. DESIGN REQUIREMENTS

The dimensions of the fish pass structure, supply of water and grade shall be suitably adjusted to reduce turbulence in the compartments to the point where the fish variety would not become fatigued. The structure entrance shall be well submerged at all stages of water when the fish are seeking ascent through it. If the flow from a fish pass is small compared to flow in the river, an auxiliary water supply may be required to increase the velocity of flow out of the fish entrance for the purpose of attracting the fish more readily.

10.1 Vertical Slot Baffle Type

Depending on rate of migration, the pool capacity shall be adequately provided to reduce fish stress. The slot width shall be adjusted in relation to the sizes of fish. A minimum 300 mm slot width shall be provided for free entry of big fish. For large slot widths, larger pool size shall be provided to cushion and dissipate the energy of the increased rate or flow passing through the slot.

10.2 Pool Capacity

The pool capacity may be determined by using the following expression (IS: 13877 – 1993):

Eq. 1

$$V = \frac{Cv}{60r}$$

where V is the pool capacity in m^3, v is the volume required per fish in m^3, C is the fish pass capacity in numbers of fish per hour and r is the rate of ascent in number of pools per minute. The size of fish pass shall depend on the following:

a. Available volume of water

b. Number of fish expected to use the fish pass

c. Size, speed and kind of species which will use the fish pass.

For bigger sizes of riverine fish, the rate of migration through the fish pass structure may be considered in the range of 2 to 5 minutes per 300 mm rise of fish pass.

10.3 Head Difference between Pool

The head loss per pool may be varied to decrease flow and check turbulence. For smaller pools (approximately 1800 mm x 2400 mm), the head per baffle shall be 200 mm, while for larger pools (3000 mm x 3600 mm), the elevation difference between pools may be kept 300 mm.

10.4 Fish Pass Gradient

The fish pass gradient shall preferably be 1:10 so as to ensure a flow velocity not exceeding 1800 mm/s in any portion of the fish pass structure. Generally, an average velocity of 400 to 900 mm/s in each pool shall be acceptable.

10.5 Baffles

To check the number of baffles required, a set of headwater and tailwater curves shall be drawn (Figure 8). To do this, the elevations of the water surface at the upstream end of the fish pass and the downstream end shall be determined from the water surface profiles at various rates of discharge (Figure 9). The maximum distance between these two curves shall guide in deciding the number of baffles required to be placed in the fish pass to overcome this head.

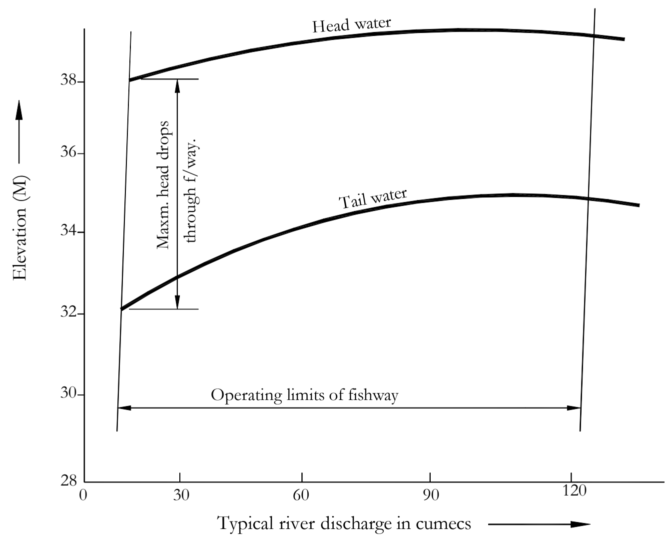


Figure 8: Typical stage discharge curve (IS: 13877 – 1993)

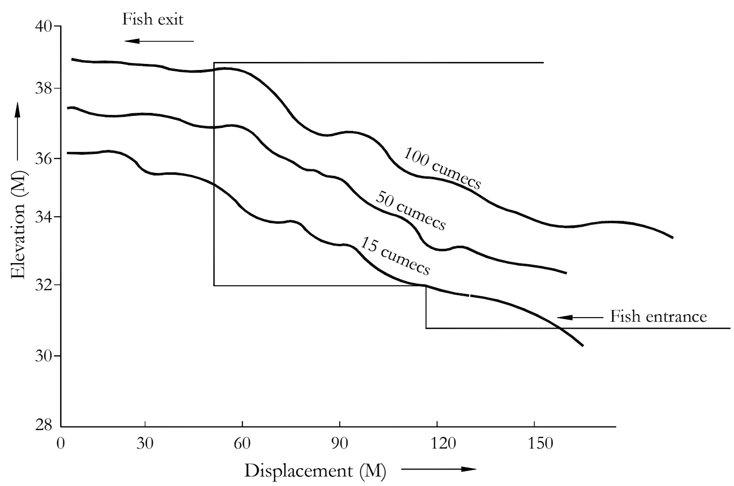


Figure 9: Typical water surface profile (IS: 13877 – 1993)

10.6 Pool

Pool depths shall usually vary from about 1 m at small ladders to about 2 m at large ladders. At weirs on small streams, pools may be from 1.25 m to 2.5 m wide and 2 to 3 m long. At weirs on large rivers, pools may be 6 m to 12 m wide, but never more than 6 m long.

10.7 Length of Fish Pass

The length of the fish pass shall be considered from the point of entrance downstream to a place above the diversion structure where relatively calm water with low velocity is available, which will not sweep away the fish passing through the fish pass.

10.8 Regulation

The upper end of the fish pass shall be provided with regulating arrangements so that the quantity of fish admitted can be controlled. Otherwise, the compartments may be unduly flooded during high water periods and would suffer from insufficient supplies during low water periods.

Automatic regulation may be provided where the variation of water level is not too great by the provision of a series of compartments with apertures through the bottom of the dividing partitions, the dimensions of the apertures being reduced in each successive partition commencing at the inlet.

11. DESIGN OF FISH BYPASS SYSTEM

The fish bypass system shall consist of one funnel which can move up and down, a metal screen to control debris, a pool type structure at the level of gate and an outlet located approximately 1 m above the river bed. The riparian flow shall also be released through this system. Considering the nature of the cold water fishes, the water drop shall be minimized, and fish shall be provided natural condition to the extent possible. Attraction mechanism such as electric light shall be provided at the entrance area.

PART 2F – INTAKE

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Part

2F

Intake

1. PURPOSE

Part 2E of the *Design Guidelines for Headworks of Hydropower Projects* provides guidance for the design of river intakes for headworks of run-of-river hydropower projects in Nepal. The guidelines are intended to ensure safe and economical design of these structures with due consideration of relevant issues, particularly those arising from conditions typical to Nepal.

2. SCOPE

The guidelines discuss the design of intakes considered suitable for run-of-river hydropower projects in Nepal. These intakes include the side intake, frontal intake and drop intake.

The guidelines cover the design philosophy and principles of the different types of intakes and provide guidance on their selection. They discuss the hydraulic design of the various components of intake structures including trash racks. They also deal with stability analysis and structural design of these components.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Contraction coefficient	Coefficient considering the effect of shape and form of piers and abutments on the approaching flow to the intake opening.
Discharge coefficient	Coefficient considering the discharging capacity of the intake opening.
Drop or trench intake	Intake structure consisting of a trough trench and trash rack over it, constructed across the river to entrap its entire minimum flow.
Free flow intake	Intake whose crest (invert) is not submerged in downstream tailwater.
Frontal intake	Intake located on the river bank with its longitudinal axis is parallel to the axis of the river flow.

Gate slots	Vertical grooves left in abutments and piers for vertical motion of gates.
Intake	Structure where the water to the power plant is abstracted or separated from the river flow.
Intake opening	Clearance for passing the discharge through the intake.
Plugging coefficient	Coefficient considering clogging capacity of the trash rack openings against floating materials on the water surface.
Rack velocity	Velocity of the water through the openings of the trash rack.
Service platform	Slab placed over the intake abutment and piers for operation and maintenance of trash racks and gates.
Side or lateral intake	Intake structure located on the river bank, usually perpendicular to the axis of the river flow.
Specific discharge	Discharge per unit length of the trash rack of the intake.
Submerged intake	Intake whose crest is submerged in the downstream tailwater.
Transition zone	Section of flow where its pattern changes from one regime to another.
Transparency coefficient	Coefficient accounting for the spaces left between the trash rack bars.
Trash rack	Perforated metallic structure composed of steel bars, angle or channel section to placed before the intake to prevent entry of floating materials, debris, etc. into the water conveyance system.
Velocity coefficient	Coefficient considering the flow capacity of the intake opening.
Vortex	Circulation vertical motion of the flow at the entrance of intake.

4. DESIGN OBJECTIVE

Intakes of run-of-river hydropower projects shall be designed to draw the desired quantity of water, limited to design discharge, from the river under controlled conditions. The design shall result in an intake arrangement that:

a. Minimizes hydraulic losses.
b. Prevent formation of air vortices.
c. Minimizes sediment entry.
d. Prevents floating debris, trash and ice from entering the water conveyance system.

5. SCOPE OF DESIGN

The design objectives enumerated in Section 4 shall be achieved through proper hydraulic and structural design of the intake structure and its components. Generally, the design shall entail the following activities:

a. Selection of suitable intake.
b. General arrangement of the intake.
c. Hydraulic design, stability and stress analysis and structural design of the structure.
d. Hydraulic design, stress analysis and structural design of the trash rack.
e. Selection of raking arrangements.

These design activities shall be carried out based on the principles and procedures discussed in the following sections.

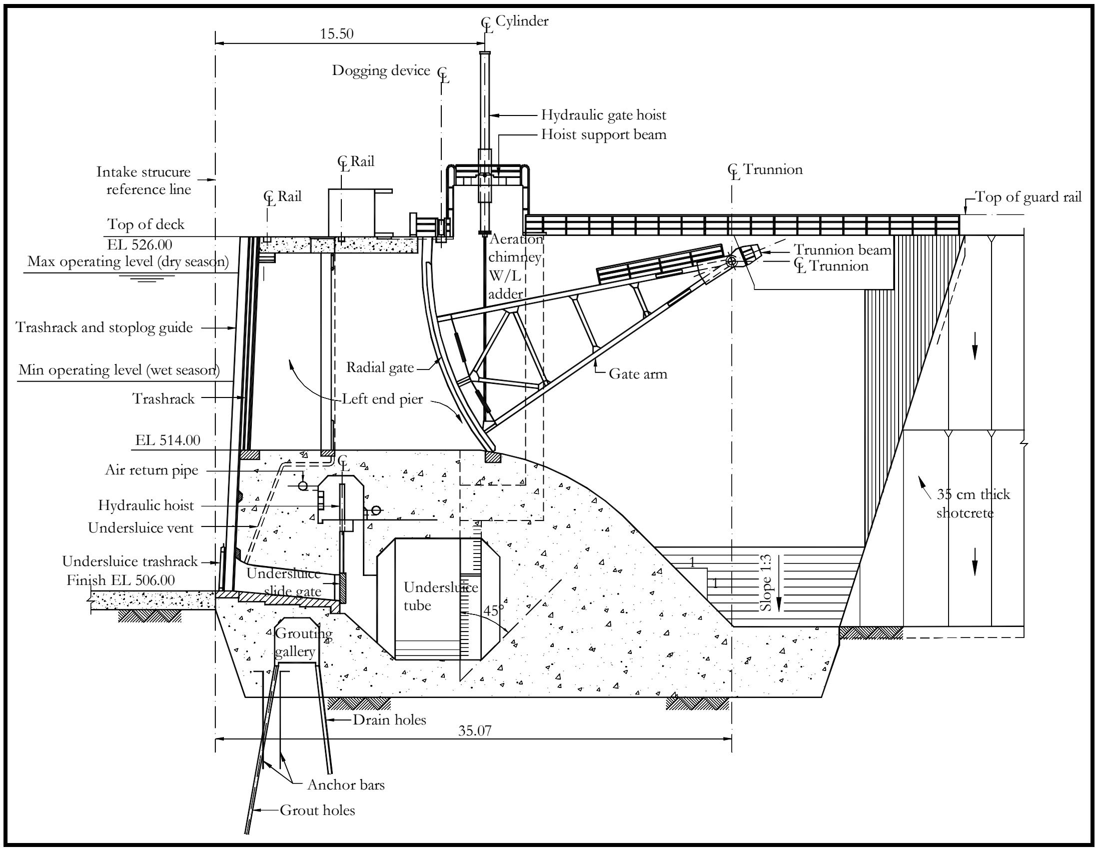

Figure 1: Intake of Kali Gandaki "A" Hydroelectric Project, Nepal (NEA, 2002)

6. DESIGN PHILOSOPHY

The intake shall be designed to be functional, hydraulically efficient, structurally optimal, economically viable and practical in operation and maintenance.

6.1 Functionality

The intake design shall ensure uninterrupted supply of the required quantity of water into the water conveyance system at all times. This requirement shall particularly be met during periods of floods when the large amounts of boulders, trash and debris carried by Nepali rivers could block or choke the trash rack, thereby forcing reduction in power generation.

6.2 Hydraulic Efficiency

The intake water passages shall be hydraulically efficient to minimize head losses. For this purpose, the forms and dimensions of the intake water passages and its other components, including piers and trash racks, shall, as far as possible, ensure smooth and streamlined flow hydraulics. The design shall aim at achieving gradual transformation of the static head to the conduit velocity and preventing formation of air-entraining vortices under pressure flow conditions.

6.3 Structural Optimality

The intake structure shall be stable under the action of the worst combination of loads likely to be act on it. Trash racks and gates provided at the intake shall be structurally safe so that power outages resulting from their breakage due to the impact of boulders and timbers transported by Nepali rivers during floods can be prevented.

6.4 Economic Construction

While satisfying hydraulic efficiency, the intake design shall also ensure that the resulting structure can be constructed economically. For small intakes, very efficient hydraulic forms shall be adopted if the net present value of the resulting reduction in operating head losses outweighs the incremental construction cost associated with the form.

6.5 Safe and Practical Operation and Maintenance

The intake shall be adaptable to safe and practical operation. The intake shall be equipped with easy raking arrangements to eliminate the need for reduction in power production to facilitate raking of the trashrack, epsecially in steep rivers with potential for flash floods and large trash. The intake design shall provide for safe working platforms and adequate facilities for storage and removal of trash removed from the trash rack.

7. TYPES OF INTAKES

Generally, one of the following types of intakes shall be used for run-of-river hydropower projects:

a. Side (or lateral) intake.

b. Frontal intake.

c. Drop (or trench) intake.

Functionally, intake also can be divided as free-flow type intake and pressure orifice type depending on type of operation required for the intake.

7.1 Side Intake

A side intake shall be used to draw water from the river through an intake structure located on the riverside (Figure 2). Its longitudinal axis shall usually be aligned perpendicular to the axis of the river. It shall normally be sited immediately upstream of the diversion structure.

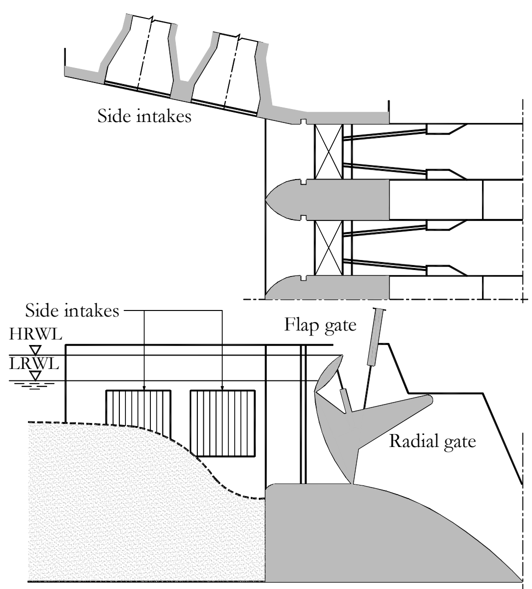

Figure 2: Typical arrangement for side intake

7.2 Frontal Intake

Like the side intake, a frontal intake shall also withdraw water from the river through an intake structure located on the river bank (Figure 3). However, its longitudinal axis shall generally be aligned parallel to the axis of the river flow. Depending on river bank conditions, the intake may be placed slightly upstream, along or downstream of the axis of the diversion structure.

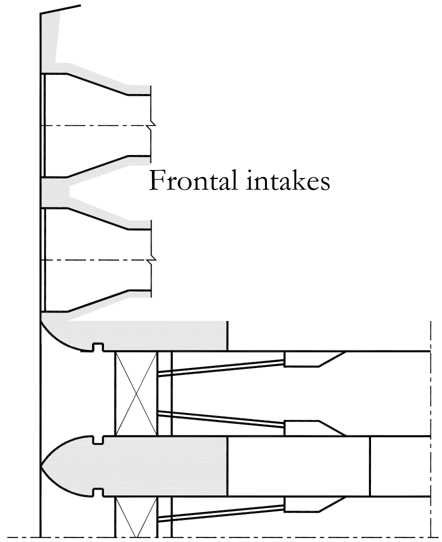

Figure 3: Typical arrangement for frontal intake

7.3 Drop (Trench) Intake

The drop intake shall form an integral part of a diversion structure (Figure 4). It shall consist of a trench-shaped intake gallery constructed in the river bed to entrap the river flow. A sediment trap trench may be provided upstream of the intake gallery to trap bed sediments. A trash rack shall be provided over the intake, often at the same level as the initial riverbed. The intake may be furnished with flat upstream and downstream aprons.

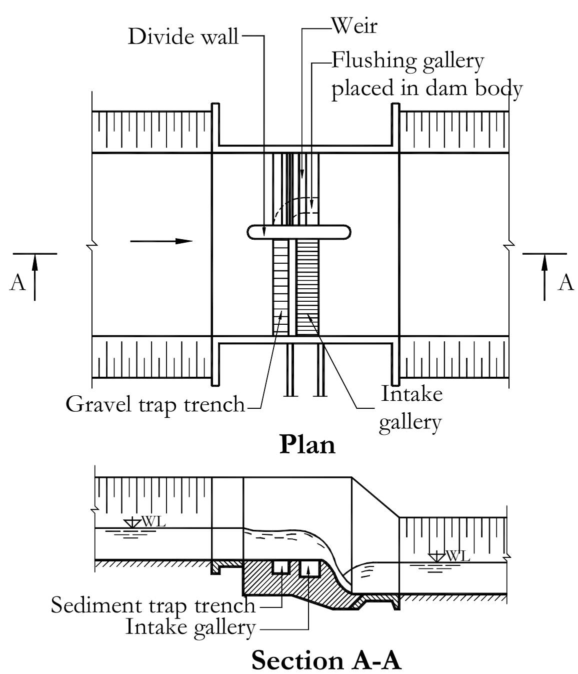

Figure 4: Typical arrangement for drop intake

8. SELECTION OF TYPE OF INTAKE

Of the intake options discussed in Section 7, the most suitable type of intake for a particular site shall be selected considering the following factors:

a. Nature of river.

b. Nature and scale of hydropower development.

c. Sediment, trash and debris content.

d. Construction considerations.

e. Operation and maintenance considerations.

The type of intake selected based on the above considerations should generally be verified through model studies.

8.1 Nature of River

Side intake may be used on all types of rivers, ranging from mild sloping silt- and sand-bed rivers to steep boulder bed-rivers or step-pool type of rivers. The use of drop intakes shall generally be limited to small hilly rivers which witness flash floods under heavy rainfall, high velocities of flow capable of transporting large quantities of sediments, floods of sufficient duration exceeding the mean discharge 10 to 20 folds or sudden muddy flows.

8.2 Nature and Scale of Hydropower Development

Side intakes may be used for any type of run-of-river hydropower development. However, frontal intakes may be preferred for low head plants where minimization of head losses commonly associated with other intakes is essential for optimal generation from the plant. Owing to their inherent additional head loss compared with side or frontal intakes, drop intakes shall generally be limited to small hydropower plants on small streams where the substantially lower construction cost of these intakes can justify the higher head loss.

8.3 Sediment, Trash and Debris Content

As their obliquity of with the river axis reduces entry of sediments and trash, side intakes shall generally be preferred over other intakes for Nepali rivers which carry large amounts of sediments, trash and debris during monsoon. This shall especially be the case when the intake can be located on the downstream end of an outer curve of a sand and gravel-bed river where secondary currents reduce the influx of sediments to the intake. In the boulder stages of rivers where rolling boulders may damage the intake foundation and trash rack, a side intake may still be used by locating the intake in a protected area. Side intakes shall, however, be used in conjunction with a gated sluice to ensure that bed load is not deposited in front of the intake.

A frontal intake located next to a free overflow section may be used in rivers with floating debris and bed load. This arrangement may be considered if the water levels at the intake and the flow velocity towards the overflow section can generate secondary currents capable of guiding floating debris over the weir and the bed load away from the intake. In this case, undersluices shall be provided to obtain bed control at the intake. Where the above arrangement is not possible, frontal intakes on sediment-laden rivers may be used only for low head hydropower plants in which the relatively large sediments flowing past the intake are not likely to damage the turbine.

Drop intakes shall generally be avoided in rivers with high sediment content because the sediment content in the abstracted water will be high as this water is drawn from the bottom of the water column where the sediment concentration is highest. In steep rivers, the trash rack to the drop intake may also be prone to damage from large boulders passing over it.

8.4 Construction Considerations

The side intake may be the most convenient for construction as it is usually constructed on dry land on the bank of the river. This advantage may also hold for frontal intakes located at a certain distance from the diversion structure; however, as pointed out in Section 8.3, this arrangement may not be suitable for restricting sediment entry.

8.5 Operation and Maintenance

Considering the large amount of trash and debris carried by Nepali rivers during floods, side intakes shall generally be preferred over other types of intakes for the following reasons:

a. Ease of trash handling, gate and stop log operation and general maintenance.
b. Lower maintenance cost due to reduced likelihood of trash rack damage.
c. Safety of operators.

Drop intakes shall not be adopted in rivers in which the mean flow remains relatively high throughout the wet season. Where such conditions exist, the intake may remain inaccessible for repair or cleanup for long periods in the case of clogging or damage of the trash rack, gravel flushing arrangements or the intake gate.

9. GENERAL ARRANGEMENT

The general arrangement of the intake shall be decided considering the following primary factors:

a. Topographical features of area.
b. Type of development, i.e. simple run-of-the river or pondage run-of-river project.
c. Proposed project configuration behind intake.
d. Content and nature of sediment in the river.
e. Construction planning.
f. Compatibility and integrity of intake with other headworks components.

Hydraulic model studies may be necessary under special conditions.

10. DESIGN OF SIDE AND FRONTAL INTAKES

The design of side and frontal intake structures shall include their hydraulic design, stability analysis and structural design. Design of trash racks for these intakes shall be performed in accordance with provisions of Section 12.

10.1 Typical Components

Side and frontal intakes shall typically consist of the following components:

a. A trash rack supporting structure.
b. Intake opening for permitting entry of water from the river.
c. Gate slot for closing intake openings / stop log grooves.
d. Breast walls for control of the flow during flood season.
e. Piers for dividing intakes with large horizontal spans into two or more sections.
f. Service platform for operation of gates and stop logs, trash handling and general maintenance.

10.2 Hydraulic Design

The hydraulic design of a side or a frontal intake shall primarily consist of fixing its intake invert level, selecting profiles for its entrance and piers and proportioning its weir.

10.2.1 Intake Invert Level

The invert level of the intake shall be fixed considering the sediment content in the river flow and previous design and construction experience. Generally, this invert shall be 1.5 to 2 m above the undersluice crest level, according to site condition, to prevent entry of bed sediments into the intake opening due to turbulence in sluice bay flow.

10.2.2 Intake Opening

The intake weir shall be designed as a broad-crested weir with submerged or free flow. The distinction between these weirs shall lie in the relative magnitudes of the critical depth of flow on the weir crest, h_{cr}, and the downstream depth of submergence, h_s, of the weir. If $h_{cr} > 1.25h_s$, the weir shall be designed as a submerged weir; however, it shall be designed as a free flow weir if this condition is not met.

10.2.3 Submerged Intake Weir

Submerged intake weirs (Figure 5) shall be designed using the equation (Zhurablov, 1975)

Eq. 1 $$Q = \delta \varepsilon \varphi B h \sqrt{2gZ_0}$$

where Q is the design discharge in m^3/s, δ is a coefficient whose value depends on the character of flow approaching the weir, ε is the coefficient for lateral flow contraction, φ is the velocity coefficient, B is the length of weir crest in m, h is the flow depth at the weir crest in m, g is the acceleration due to gravity in m/s^2 and Z_0 is the difference in the upstream and downstream water levels, including approach velocity in m.

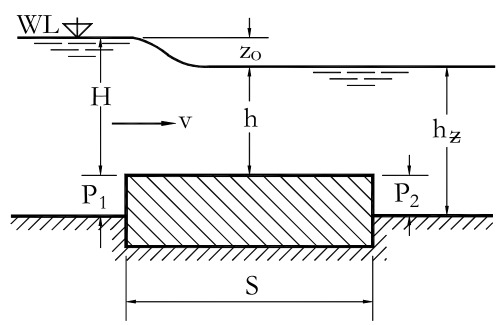

Figure 5: Submerged intake weir

The value of δ shall depend on the angle α between the longitudinal axis of the intake and the axis of the river flow. Values of δ for typical values of α are presented in Table 1.

Table 1: Values of coefficient δ for different values of α

α	0°	30°	45°	60°	75°	90°
δ	1	0.97	0.95	0.93	0.90	0.86

The coefficient for lateral flow contraction ε shall be computed from the equation

Eq. 2 $$\varepsilon = 1 - a_{cont} \frac{H}{B+H}$$

where a_{cont} is the coefficient of contraction depending upon the form of piers, taken equal to 0.20 for rectangular piers, 0.10 for semi-circular piers and 0.05 for elliptical piers, and H is the head over the weir crest in m.

Likewise, the values of the velocity coefficient φ for different conditions of flow shall be based on Table 2.

Table 2: Velocity and discharge coefficients for broad crested weirs

Condition of flow	φ	C_d
Absence of hydraulic friction	1.00	0.385
Elliptical form of crest and pier	0.95	0.365
Circular form of crest and pier	0.92	0.350
Rough form of crest and pier	0.88	0.320
Sharp form of crest and pier	0.85	0.320
Worse hydraulic conditions	0.80	0.300

(Source: Zhurablov, 1975)

The value of *B* shall be determined iteratively using Eq. 1 and Eq. 2. For this purpose, an initial value of ε shall be assumed, and the iteration shall be repeated till the computed and assumed (or updated) values of ε converge to acceptable limits. For good performance, the ratio of *B* and *h* shall generally be maintained between 1.2 and 1.5.

10.2.4 Free Flow Intake Weir

The discharge over the broad-crested weir for free flow conditions shall be determined by the formula (Zhurablov, 1975)

Eq. 3 $$Q = \delta\varepsilon C_d B\sqrt{2g}H_o^{3/2}$$

where C_d is the discharge coefficient obtained from Table 2 and H_o is the head over the weir crest, including the approach velocity, in m.

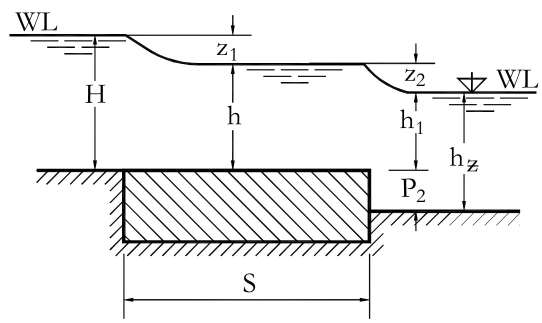

Figure 6: Free flow intake weir

10.2.5 Submerged Flow Under Gates

The discharge over a gated intake weir under submerged flow conditions (Figure 7) shall be determined as (Zhurablov, 1975)

Eq. 4 $$Q = \mu a B\sqrt{2g(H_o - h_z)}$$

where h_z is the depth of flow at the section where the contractioned flow is observed through the unsubmerged flow and μ is the discharge coefficient ranging between 0.60 and 0.85.

The flow depth h_z can be obtained from the equation

Eq. 5 $$h_z = \sqrt{h_o^2 - N\left(H_o - \frac{N}{4}\right)} + \frac{N}{2}$$

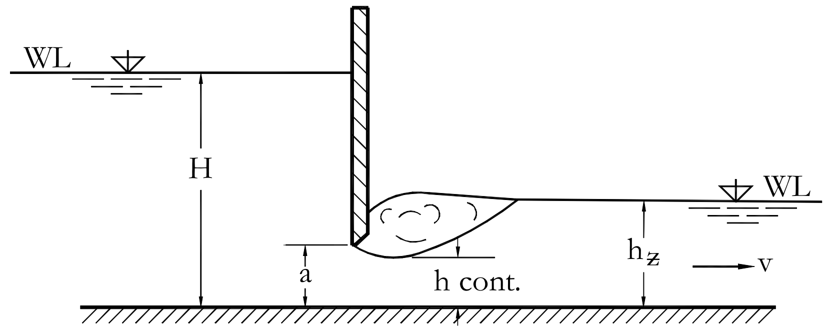

Figure 7: Gated intake weir under submerged flow conditions

where

Eq. 6 $$N = 4\mu^2 a^2 \frac{h_o - h_{cont}}{h_o\, h_{cont}}$$

in which h_o is the downstream normal depth of flow during submerged flow and h_{cont} is the flow at the contractioned section just after the gate downstream.

10.2.6 Free Flow under Gates

Under free flow conditions, the discharge over a gated intake weir () shall be found using the relation (Zhurablov, 1975)

Eq. 7 $$Q = \varphi\varepsilon' aB\sqrt{2g\left(H_o - \varepsilon' a\right)}$$

where φ is a velocity coefficient ranging between 0.95 and 0.97 for openings without crests and between 0.85 and 0.95 for openings with elevated crest, ε' is a coefficient for vertically flow contraction that depends on the ratio of opening height to the depth of flow before the gate and *a* is the gate opening, and

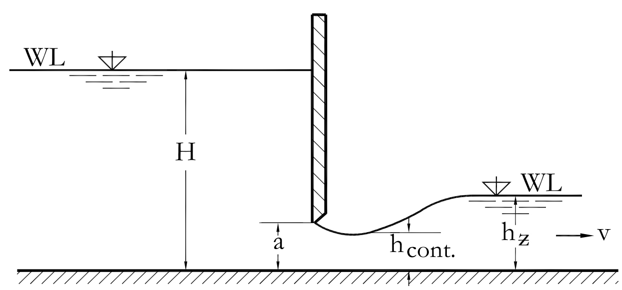

Figure 8: Gated intake weir under free flow conditions

Eq. 8 $$H_o = H + \frac{v_0^2}{2g}$$

where H is the static head and v_o is the approach velocity.

For vertical plane gates, ε' shall range between 0.615 for a/H = 0.10 to 0.69 for a/H=0.70. For deep openings closed by gates with curved surface (e.g. a radial gate), the discharge coefficient $\varphi\varepsilon'$ in Eq. 7 shall depend upon the inclined angle β and can roughly be taken as 0.74 for $\beta = 63°20'$ and 0.84 for $\beta = 45°$.

10.2.7 Approach Apron

The intake approach apron shall not be placed closer than 30 percent of the intake height measured from the lower edge of the intake invert.

10.2.8 Intake Piers

At intakes with large horizontal spans, vertical reinforced concrete piers shall be provided to divide the intake into two or more sections The piers may be used to support the trash racks, leaving a flat clear rack for easy access and cleaning. In some cases, the pier noses may extend beyond the trash racks to allow stop logs to be installed in grooves in front of the racks. For the latter arrangement, the rack cleaner shall fit into the spaces between the piers; however, this arrangement may result in trash collecting in large quantities adjacent to the piers.

Intake piers shall be designed as an optimal compromise between smooth flow hydraulics and structural design convenience. The nose of the vertical pier shall preferably be rounded or may conform to the shapes (Figure 9) streamlined about the required structural section. The trailing edge of the piers, too, may use these or other efficient forms; however, sharp 90º corners, which have often been found to be as efficient as the more complex shapes, may be adopted for simplicity in construction.

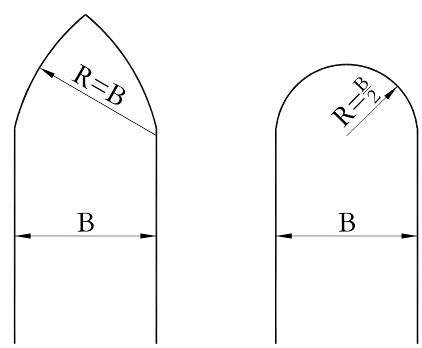

Figure 9: Typical pier shapes

10.2.9 Intake Losses

The dimensions and form of the intake shall be made with regard to limiting the head loss, without making the intake too expensive to construct. Intake head losses shall be computed as (USBR, 1978)

Eq. 9
$$H_i = K\frac{V_n^2}{2g}$$

where H_I is the intake head loss in m, K is the intake loss coefficient, V_n is the normal velocity through intake in m/s and g is the acceleration due to gravity in m/s^2.

As the intake is usually a smooth construction of short length, friction losses shall usually be neglected in the intake loss calculation. Therefore, the loss coefficient shall usually consist of two parts, namely

Eq. 10
$$K = K_i + K_t$$

where K_i is the intake loss due to sudden contraction in flow from the reservoir as it passes the trash racks and piers and K_t is the gradual contraction losses as the flow follows the transition part of the intake into the intake gate or into the headrace where the cross section becomes constant. Some approximate values for the two types of losses are given in Table 3 and Table 4.

Table 3: Typical values of K_i

Shape	K_i
Bell mouth	0.03 - 0.05
Slightly rounded	0.12 - 0.25
Sharp cornered	0.50

(Source: USBR, 1978)

Table 4: Typical values of K_t

Cone angle	K_t
30º	0.002
45º	0.04
60º	0.07

(Source: USBR, 1978)

10.2.10 Transitions

In order to obtain hydraulically efficient design of intake transitions between intake and approach canal, the transition shall be designed to satisfy the following requirements:

a. Transition or turns shall be made about the centre line of mass flow and shall be gradual.
b. Side walls shall not be expanded at a rate greater than 5º to 7º from the centre line of mass flow.
c. All slots or other necessary departures from the neat outline shall normally be outside the transition.

The upstream transition shall be designed in accordance with the topographical, geological and hydrological conditions of the site. The downstream conditions shall be designed according to the flow regime from the intake to the approach canal transition.

10.3 Stability Analysis

Intake structures shall satisfy all stability requirements defined in Part 2B of the guidelines for diversion structures. They shall be stable even under dewatered conditions.

10.3.1 Design Loads

The following loads shall be considered for the stability analysis of intake structures:

a. Dead load.
b. Headwater and tailwater pressures.
c. Uplift pressure.
d. Earthquake forces.
e. Earth pressure.
f. Silt pressure.
g. Wind pressure.
h. Wave pressure.
i. Thermal loads.
j. Reaction of foundations.

The magnitudes of these loads shall be computed based on procedures discussed in Part 2B of the guidelines.

10.3.2 Load Conditions

Intake structures shall be designed for the load conditions listed in Table 5.

Table 5: Load conditions for stability analysis

Condition	Description
Usual	• Pool at full supply level • All gates closed • Conduit empty
Extreme	• Pool at full supply level • All gates closed • Conduit empty • Earthquake

10.4 Structural Design of Intake Piers

The structural design of intake piers, where provided, shall be performed according to the provisions for design of piers of diversion structures presented in Part 2B of the guidelines. In doing so, only load cases and conditions applicable to the intake piers shall be used.

11. DESIGN OF DROP INTAKE

The design of drop intakes shall involve sizing of the intake gallery and the sediment trap trench.

11.1 Intake Gallery

Design of the intake gallery shall consist of fixing its cross-section and length (Figure 10).

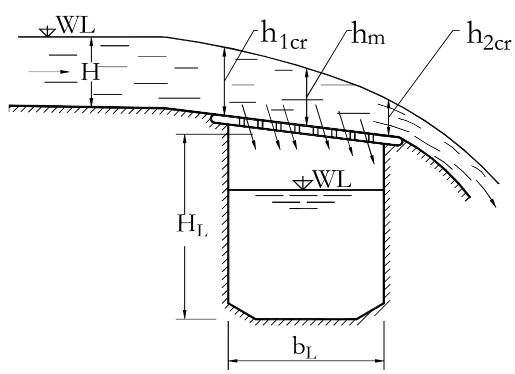

Figure 10: Intake gallery of drop intake (Zhurablov, 1975)

Neglecting the sediment trap trench, the rack part shall be designed to pass the discharge Q_r given by

Eq. 11
$$Q_r = (1.25 \text{ to } 1.5)Q_c$$

where Q_c the canal discharge in m^3/s, including the discharge for sediment flushing in the sediment trap.

The plan dimensions of the intake shall be obtained from the relation (Zhurablov, 1975)

Eq. 12 $$Q_c = C_t \mu C_p l_r b_r \sqrt{2gh_m}$$

where C_t is the transparency coefficient, μ is a coefficient ranging from 0.60 to 0.65 for s = 0.1 and 0.55 to 0.60 for s = 0.2, C_p is the coefficient normally taken equal to 0.90, l_r is the length of the rack opening in m, b_r is the width of the rack opening in m and h_m is the depth at the middle of the rack in m.

The transparency coefficient C_t shall be computed using the equation

Eq. 13 $$C_t = \frac{t}{t + \delta}$$

where t is the opening between trash rack bars and δ is the thickness of the rack bars.

The depth of flow h_m at the middle of the rack shall be determined using the following empirical relationship:

Eq. 14 $$h_m = 0.81 \frac{h_{1cr} + h_{2cr}}{2}$$

where h_{1cr} is the critical depth at the beginning of rack for the flow depth H before the trench and h_{2cr} is the critical depth at the end of the rack after abstraction of required discharge through the intake.

The critical depths h_{1cr} and h_{2cr} shall be computed using the equations (Zhurablov, 1975)

Eq. 15 $$h_{1cr} = 0.47 q_1^{2/3}$$

and

Eq. 16 $$h_{2cr} = 0.47 q_2^{2/3}$$

where q_1 and q_2 are the specific discharges at the beginning and end of the rack, respectively, computed as (Zhurablov, 1975)

Eq. 17 $$q_1 = \frac{Q_r}{l_r}$$

and

Eq. 18 $$q_2 = \frac{(Q_r - Q_c)}{l_r}$$

In order to fix the magnitudes of the two unknowns l_r and b_r in Eq. 12, an initial value of rack length l_r may be obtained from the expression (Zhurablov, 1975)

Eq. 19 $$l_r = \frac{Q_c}{q_r}$$

where q_r the specific discharge per unit length of the rack, generally taken between 0.5 to 1.0 m^2/s or more. For this value of l_r, the width b_r shall be obtained from Eq. 12. For proper dynamic functioning of the intake, b_r shall generally be limited to 2 to 2.5 m to avoid very heavy trench dimensions. For this purpose, b_r may be recomputed using a different value of q_r.

11.2 Gravel Trap Trench

A gravel trap trench shall be provided just before and parallel to the intake gallery to avoid entry of bed sediments into the latter (Figure 4). The trench shall have a cross-section of 600 x 600 mm and shall be covered by a trash rack with spacing of rack bars 1.5 to 2.5 times larger than that for the trash rack over the intake gallery. The trench shall be connected to a flushing gallery, which could pass through the diversion structure, to continuously flush the collected sediments. A gate shall be provided before the flushing gallery to control of flow through the sediment trap trench and stop its functioning when required.

12. TRASH RACKS

Trash racks shall be provided at the intake entrance to prevent the entry of any trash, such as grass, leaves, trees, bushes, timber, suspended sediments or rolling boulders, which would not pass easily through the smallest opening in the turbine runner. In cold areas, the trash rack shall also check the entry of ice sheets.

12.1 Types of Trash Racks

Generally, three types of trash racks, namely Type 1, Type 2 and Type 3, shall be used with run-of-river intakes.

Type 1 trash racks shall consist of removable section racks that are installed by lowering the sections between side guides or grooves provided in the trash rack structure These are generally side bearing type.

Type 2 trash racks shall consist of removable section racks in which the individual sections are placed adjacent to each other laterally and in an inclined plane to obtain the desired area. To prevent the rack sections from being displaced, the individual sections are secured in place with bolts located above the water line.

Type 3 trash racks shall consist of section racks which are bolted in place below water line.

12.2 Selection of Trash Rack Type

The selection of the type of trash rack for a particular intake shall be based on the following considerations:

a. Accessibility for painting or replacement.
b. Size and quantity of trash expected.
c. Requirement of raking.

Type 1 trash racks shall be used for all major trash rack installations where a portion of rack is deeply submerged. Racks of Type 2 shall be used for intakes where a single rack section extends from the water surface to the bottom of rack. Likewise, Type 3 racks shall be used where power driven cleaning rakes are provided for raking.

12.3 Hydraulic Design of Trash Racks

The hydraulic design of trash racks shall consist of determining the shape of the trash rack structure, inclination of racks and geometry of rack bars.

12.3.1 Shape of Trash Rack Structure

The shape of the trash rack structure shall be chosen to meet the requirements of the headworks layout and head losses. Generally, a straight trash rack structure shall be opted for ease of construction.

12.3.2 Inclination of Racks

The inclination of racks shall be fixed based on practical consideration related to the raking operation. Except for guided racks, racks shall be installed in a slight inclination so that trash does not roll along the rack during upward raking. For manual raking, the slope shall be 1 vertical to 0.33 or 0.5 horizontal. Where mechanical raking arrangement is provided, the slope of the racks shall be kept at 10º to 15º with the vertical unless otherwise specified by the manufacturer of the raking equipment.

12.3.3 Rack Velocity

The velocity of flow through the rack structure shall be limited to 0.75 m/s for small units with closely set rack bars or at intakes where manual raking is provided. A velocity up to 1.5 m/s shall be permitted at large units with wider spacing of rack bars and where mechanical cleaning of racks is provided.

12.3.4 Rack Bar Geometry

From hydraulic considerations, a streamlined rounded and tapered rack bar shape shall be desirable. However, considering the higher cost of these bars and the possibility of jamming of trash between them, simple rectangular bar type racks may normally be used, provided such bars do not result in excess head losses.

12.3.5 Losses at Trash Racks

Head loss at trash racks shall be calculated from the formula (IS: 11388 – 1995)

Eq. 20

$$h_r = K\frac{v_r^2}{2g}$$

where K is the trash rack loss coefficient in m, v_r is the net velocity of flow through trash rack, computed on gross area, in m^2 and g is the acceleration due to gravity in m/s^2.

In most cases, the value of K may be approximated using the empirical relation (IS: 11388 – 1995)

Eq. 21

$$K = 1.45 - 0.45R - R^2$$

where R is the ratio of the net area through the rack bars to the gross area of the racks and their supports.

Alternatively, the head losses may be computed using the following formula (IS: 11388 – 1995):

Eq. 22

$$h_r = k\left(\frac{t}{b}\right)^{1/8}\frac{v^2}{2g}\sin\alpha$$

where h_r is the loss of head through racks, t is the thickness of rack bars, b is the clear spacing between rack bars, v is the velocity of flow through the trash rack computed on gross area, α is the angle of bar inclination to the horizontal and k is a factor depending on bar shape, determined in accordance with Figure 11.

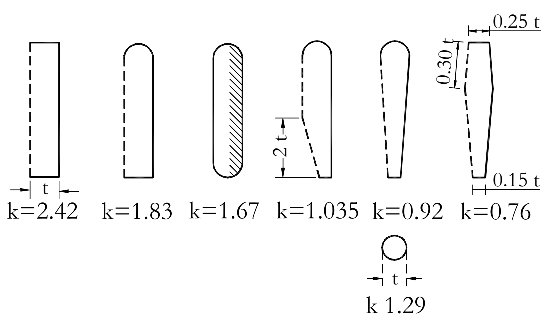

Figure 11: Values of trash rack coefficient for different bar shapes (IS: 11388 – 1995)

The value of h_r computed from the above equations shall be multiplied by a factor 1.75 to 2 to take care of the trash rack bracing and frame. Allowance for increase in the flow velocity between bars due to in partial clogging of racks shall also be made in the head loss estimate. In view of the large amount of trash in Nepali rivers during floods, 25 to 50 percent of the area of racks may be considered to be obstructed by trash.

12.4 Structural Design

12.4.1 General Arrangement

The trash racks shall generally consist of equally spaced vertical bars supported on horizontal members (Figure 12). The horizontal members, in turn, shall be connected to end vertical members sitting in the grooves of piers. The size of each trash rack unit shall be proportioned from consideration of hoisting/lifting capacity.

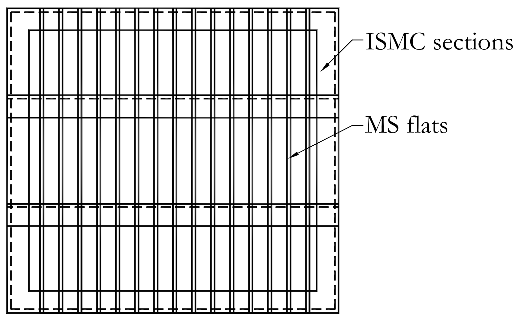

Figure 12: Metallic trash rack

12.4.1.1 Spacing of Trash Bars

The clear spacing of the vertical rack bar shall generally be 5 mm less than the minimum opening in the turbine runner blade or wicket gates. It may vary between 40 to 100 mm. In general, a close spacing shall be adopted for small turbines while a wider spacing shall be preferred for larger ones.

For Francis turbines, the spacing of trash bars shall be determined considering its specific speed, runner diameter and number of buckets It shall be about l/30 of the runner diameter for propeller or Kaplan turbines. For impulse turbines, the spacing shall not be larger than l/5 of the jet diameter at maximum needle opening; however, for small impulse turbines, a mesh screen shall be permitted.

12.4.1.2 Spacing of Horizontal Members

The spacing of horizontal members of the trash rack lie between 400 to 500 mm. The spacing shall ensure that the laterally unsupported length of trash rack bar does not exceed 70 times the bar thickness.

For intakes on most Nepali rivers, the spacing between one or two bottom horizontal members shall be considerably reduced, say between 150 to 200 mm, to prevent the rolling sediments carried by the rivers from entering the intake. This provision shall also be adopted to reduce vibrations in the trash rack structure caused by the impact of boulder.

12.4.1.3 Bar Dimensions

The thickness of trash bars for Type 2 and Type 3 trash racks shall not be less than 8 mm. For deep submerged racks, the minimum thickness shall be kept as 12 mm. The depth of trash bar shall not be more than 12 times its thickness and nor less than 50 mm.

12.4.1.4 Bearing Pads

Trash racks shall be provided with bearing pads to protect the protective coating of racks from abrasion due to in contact with the concrete grooves. The pads shall not less than 10 mm thick.

12.4.2 Materials

The trash rack shall be fabricated from structural steel. The steel shall preferably be resistant to corrosion.

12.4.3 Design Head

The design head for the trash rack shall be selected taking into consideration the intensity of trash inflow and the efficiency of racking. This head shall depend on the difference in the upstream and downstream water levels of the rack at the time of maximum clogging. Although the head is site dependent, the following guidelines may be adopted for design purposes:

a. Rack bars and their steel supports shall be designed for 25% of the total differential head to which they might be subjected if wholly clogged.
b. For intakes where complete and sudden clogging of rack is a distinct possibility, the design head for all portions of the intake shall be that resulting from complete stoppage of flow through the racks.

The designer shall exercise discretion in selecting the design head in order to arrive at a safe and economical design.

The design head for trash racks in hydropower projects in Nepali requires consideration of the heavy bed load carried by rivers in addition to floating debris during the monsoon season. As this bed load is sizable in magnitude and, therefore, difficult for racking when accumulated against trash racks, the trash rack shall be designed at two third of the maximum depth of submergence with normal permissible stresses.

12.4.4 Failure Stress

Trash rack bars shall be assumed to fail when the stress in them reaches the following value (IS: 11388 – 1995):

Eq. 23

$$\sigma = \sigma_y \left(1.23 - 0.0153\frac{L}{t}\right)$$

where σ_y is the yield stress of the bar material, L is the laterally unsupported length of the trash bar and t is the thickness of the trash bar.

The safe working stress for trash rack bars used to support flash boards shall not exceed the following value:

Eq. 24

$$\sigma = \frac{2}{3}\sigma_y \left(1.23 - 0.0153\frac{L}{t}\right)$$

12.4.5 Design of Horizontal Members

Members used as horizontal beams in trash rack sections shall not require stress reduction to compensate for lack of lateral support. These members shall be assumed to fail at yield stress, but calculations shall include stress due to dead weight of the beam members and trash rack bars. To ensure rigidity during handling, the lateral deflection of the beam members due to loads shall not exceed 1/325 of the span.

12.4.6 Stability against Vibrations

Trash racks shall be checked for resonance while operating under turbine modes, and the design and disposition of the members shall be so made that resonance does not take place. For normal conditions, the forcing frequency shall be limited to less than 0.6 times the natural frequency; however, a higher forcing frequency not exceeding 0.65 may be permitted for a short period.

12.5 Structural Details

Structural connections in the trash rack shall be designed and provided for the failure load of the structural members. All flats shall be welded to the intermediate horizontal members and the top and bottom horizontal members for better resistance to vibrations and to avoid stress concentration at the external edge of the groove. The vertical member of the trash rack shall be so arranged as to apply the load near the inner part of the rack guide.

Type 1 racks, where used in tiers, shall be equipped with dowels of sufficient size to ensure proper alignment of the racks in the guides. The guides of the trash racks shall be so proportioned that the side members get lateral support from guides after deflection to take up the clearance in the slots. The height of units of Type 1 shall be equal to the spacing of the horizontal concrete arch ribs of intake structure or its convenient fraction. For proper seating of one trash rack unit above the other, pilot shoes and pilot pins shall be provided.

12.6 Raking Arrangement

The trash rack shall be provided with suitable arrangements for removing debris at regular intervals. Continuous raking arrangements shall be made at intakes which are likely to continuously attract floating material due to an abundance of such material in the flow and due to the level of water being often near the trash rack level.

12.7 Trash Racks for Drop Intake

Racks for covering the intake gallery of drop intakes shall be fabricated from structural steel. Normally, T-shaped rack bars shall be used to prevent the sediments from plugging the openings between these bars and to permit their cleaning.

13. CONTROL GATES

Control gates shall be provided downstream of the trash rack in order to regulate the flow of water into the water conveyance system, to permit closure of the desander or the water conveyance system during dewatering for inspection or to protect the generator unit during emergencies.

13.1 Types of Gates

Control gates in the form of a vertical lift gate shall usually be provided in the water passage. This gate shall normally be suspended just above the roof of the intake from a fixed hoist, preferably removed completely from the water passage when fully open. Slide gates or wheel gates shall be used if the gates are large or operate under higher pressures. If the gate has to be designed to close in flowing water or to operate in part-open positions for long periods of time, a radial gate may be preferred despite the fact that more space is required for it.

13.2 Velocity through Gates

The location of the control gates shall be selected considering the economical gate size and the permissible velocities of flow. The permissible velocity of flow, v, through the intake gate shall be given by the expression

Eq. 25
$$v = 0.12\sqrt{2gh}$$

where h is the head from the center line of the gate to the normal water surface.

13.3 Gate Slots

Intake gate slots shall be enclosed in a structure designed to guide the water into the intake opening without side contraction. The minimum distance between the upstream edge of the gate slot and the nose shall be 0.40 times the intake opening. Where gates are located in a gate shaft, suitable transitions to the rectangular gate slot shall be provided.

13.4 Stop Logs

Stop logs or bulkhead gates shall be provided just upstream of a control gate to allow dewatering during its maintenance operations. They shall be heavy concrete or steel beams that can sink horizontally into vertical grooves in the intake piers designed to support them. As they are almost impossible to put in place in flowing water, stop logs or bulkhead gates shall never be relied on as an emergency closure facility.

PART 2G – APPROACH CANAL

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Part

2G

Approach Canal

1. PURPOSE

Part 2G of the *Design Guidelines for Headworks of Hydropower Projects* provides technical criteria and guidance for the design of approach canals for headworks of run-of-river hydropower projects in Nepal.

2. SCOPE

The guidelines cover the design of approach canal connecting the intakes and settling basin of run-of-river hydropower projects. They also include the design of gravel traps/ejectors.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Approach canal	Canal for conveying water abstracted at the intake to the settling basin.
Gravel trap/ ejector	Structure placed in the approach canal to trap all gravel and flush it into the river.
Gravel flushing	Process of flushing gravel trapped by gravel trap / ejector.

4. DESIGN OBJECTIVES

The approach canal shall be designed to convey the abstracted flow from the intake to the settling basin in a hydraulically efficient manner. Likewise, the gravel trap, if required, shall be designed to trap and efficiently flush out the coarse sediments that manage to enter the approach canal through the intake.

5. SCOPE OF DESIGN

In order to attain the objective stated in Section 4, the design of approach canals and gravel trap shall include the following activities:

a. Hydraulic design of the approach canal.
b. Hydraulic design of the gravel trap, including its flushing mechamism.
c. Selection of approach canal lining.

6. DESIGN OF GRAVEL TRAP

A gravel trap shall be required to flush out bed sediments that enter the approach canal back into the river. The necessity of a gravel trap may arise owing to faulty design of the river intake.

6.1 Location

The gravel trap may be placed below the invert level of an intake structure floor. Alternately, it may be located downstream of the intake structure in the approach canal.

6.2 Regimes of Flow

The gravel trap designed just below the invert level of a river intake structure shall always function as a pressure canal. However, it may function as a pressure or a non-pressure canal if it is placed in the approach canal after the river intake structure. In either case, its minimum opening shall not be less than 0.80 m. The opening of a gravel trap located at the river intake shall always be provided with trash racks to protect against clogging by boulders.

6.3 Hydraulic Design

6.3.1 Critical Velocity

For the size of gravels to be transported by the flow in the gravel trap, the critical velocity of flow shall be determined by

Eq. 1 $$V_{cr} = a\sqrt{d}$$

where, V_{cr} is the critical velocity of flow in cm/s, a is an empirical coefficient equal to 44.0 for particle size greater than 1 mm; d is the particle size to be transported by the flow in mm.

6.3.2 Length of Settling Trap

Based on the flow velocity in the trap and the settling time of the particle, the length of settling trap shall be obtained as

Eq. 2 $$L = t_{set}.V$$

where L is the length of settling trap in m and t_{set} is the settling time of particle given by

Eq. 3 $$t_{set} = \frac{h}{\omega}$$

where h is the depth of flow in m and ω is the settling velocity in m/s.

6.3.3 Flushing Velocity

The flushing velocity in the trap gallery (canal) may be found by the formula

Eq. 4 $$V_f = \Psi\sqrt{2gh}$$

where V_f is the flushing velocity in m/s, X is the velocity coefficient (0.60 to 0.80), g is the acceleration due to gravity in m/s^2 and h is the differential head in m.

6.3.4 Flushing Time

The time taken for flushing (emptying) the volume of water and gravel mixture from the gravel trap basin will be determined as

Eq. 5

$$T = \frac{V_0}{Q_f}$$

where T is the flushing time in s, V_0 is the volume of the gravel-water mixture in the gravel trap basin in m^3 and Q_f is the flushing discharge of the two-phase mixture at the gallery in m^3/s.

6.3.5 Type of Flushing

The flushing of the gravel-water mixture may be performed either by periodical or by continuous flushing depending upon the type of gravel trap/ejector basin designed.

6.3.6 Lining

The gravel trap shall be lined with abrasion-resistant material (refer Section 11.2.7 of Part 2B). This lining shall be terminated at the point where normal flow conditions are attained.

6.3.7 Gates and Stop Logs

Gates and stop logs for controlling the outlet of the gravel trap shall be provided nearest to its outflow. Usually, the outlet level of the trap shall be positioned above the 5-year return period flood.

7. DESIGN OF APPROACH CANAL / CONDUIT

7.1 Design Criteria

The approach canal connecting the intake structure to the settling basin shall have an optimum length chosen with due consideration to the topography of site. However, the canal shall be economically effective and hydraulically efficient for transporting the specified size of sediments passing through the intake structures.

7.2 Design Assumption

The design of the approach canal shall be performed assuming steady (sub-critical) flow regime in the canal. Under this assumption, the canal shall be designed using Chezy's or Manning's formula.

7.3 Hydraulic Design

7.3.1 Regimes of Flow

The regime of flow in an approach canal may be semi-pressure or non-pressure flow. In case of semi-pressure flow regime at the outlet of this canal, an energy dissipation arrangement shall be made towards the entrance of the settling basin to dissipate the excess energy in the flow. However, in either case, a steady flow regime shall be maintained, i.e. the mean flow velocity in the approach canal shall be considered to be constant, not changing in time and space along and/or on the canal flow section.

7.3.2 Canal Cross-section

The cross-section of an approach canal may be rectangular or trapezoidal depending upon the topographical nature of site and soil/rock characteristics. With regard to loss of energy, however, the canal cross-section shall be economically effective.

7.3.3 Hydraulic Characteristics

Based on the principle of non-silting and non-scouring velocity with efficient transporting capacity of flow with regard to sediments, either Manning's or Chezy's formula may be applied to determine the hydraulic characteristic of the approach canal. Manning's equation shall be given by

Eq. 6 $$V = \frac{1}{n} R^{2/3} S^{1/2}$$

where V is the mean flow velocity in the canal in m/s, n is Manning's rugosity coefficient, S is the longitudinal bed slope and R is the hydraulic mean radius in m, given by

Eq. 7 $$R = \frac{A}{P}$$

in which A is the flow area of cross-section in m^2 and P is the wetted perimeter in m.

Likewise, Chezy's formula shall be given by

Eq. 8 $$V = C\sqrt{RS}$$

where C is Chezy's coefficient.

As Manning's and Chezy's coefficients are sensitive to increase or decrease in flow velocity and can directly affect the size of an approach canal, the coefficients shall be chosen as to reliable values as per soil/rock characteristics of the site.

7.3.4 Canal Lining

Based on site conditions, the approach canal may be lined or unlined. In soils with relatively high permeability, the canal shall be lined along its entire length to prevent seepage losses. Depending upon the permeability of the soil mass, the lining may be performed with impervious materials like clay, clayey loam, concrete, reinforced concrete, steel sheet, etc.

7.4 Design of Conduits

When the topography of the headworks area does not offer the possibility of aligning an open canal just after the intake structure, a pressure conduit shall be used instead of an approach canal up to the settling basin. The conduit shall be designed for flow velocities in the range of 1 to 1.5 m/s. The dimension of this conduit may be found from the formula

Eq. 9 $$Q = \mu A\sqrt{2gz}$$

where Q is the discharge of the conduit including flushing discharge in m^3/s; μ is the discharge coefficient ranging from 0.60 to 0.80; A is the flow area of the conduit in m^2; g is the acceleration due to gravity in m/s^2 and z is the differential head over the conduit in m.

PART 2H – SETTLING BASIN

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Part

2H

Settling Basin

1. PURPOSE

Part 2F of the *Design Guidelines for Headworks of Hydropower Projects* provides technical criteria and guidance for the design of settling basins for headworks of run-of-river hydropower projects in Nepal. The guidelines are intended to ensure effective and economical design of these structures in consideration of the high suspended sediment content of Nepali rivers, especially during floods.

2. SCOPE

The guidelines cover the design of settling basins deemed suitable for run-of-river hydropower projects in Nepal. They deal with periodic and continuous flushing settling basins, including their inlet and outlet transitions, flushing systems and emergency spillways.

The guidelines deal with the philosophy, principles and design data requirements for the design of settling basins. They discuss the hydraulic and structural design of these structures.

3. TERMINOLOGY

Terms used in these guidelines are defined below:

Continuous flushing settling basin	Settling basin in which settling and flushing of sediments proceed simultaneously.
Emergency spillway	Structure provided with settling basin to safely spill the settling basin design discharge coming from an approach canal or an intake into the parent river, usually with the help of a chute or ogee type overflow spillway.
Flushing structure	Structure provided with settling basin to flush out the sediments deposited in it.

Lifting velocity	Vertical component of flow velocity that causes which movement of sediments in suspended condition.
Longitudinal velocity	Velocity of flow of sediment-laden water through the settling basin.
Periodic flushing settling basin	Settling basin in which sediments are allowed to settle first and are then flushed out hydraulically.
Sediment concentration	Content of suspended soil particles in unit volume of water-sediment mixture.
Sediment transport capacity of flow	Quantities of sediments contained in 1 cu. m. of flowing water per unit time.
Settling basin	Structure provided to remove suspended sediments from the water abstracted from the river.
Settling velocity	Velocity with which suspended sediments sink in water under the action of gravity.
Tranquilizer	A kind of filter wherein the flow is forced to be distributed across the settling basin cross-section with a certain head loss.

4. DESIGN OBJECTIVES

Settling basins shall be designed to ensure that the water entering the water conveyance system is free of sediments that can damage the penstock and turbines runners due to abrasion. This shall be achieved by reducing 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.

5. SCOPE OF DESIGN

The design objective stated in Section 0 shall be achieved through proper hydraulic and structural design of the settling basin and its associated structures. Normally, the design shall consist of the following activities:

a. General arrangement of the settling basin, its flushing structures and its inlet and outlet transitions,.
b. Hydraulic design of the settling basin, its flushing structures and its inlet and outlet transitions.
c. Stability analysis, stress analysis and structural design of the settling basin.

These activities shall be carried out based on the principles and procedures discussed in the following sections.

6. DESIGN PHILOSOPHY

The settling basin shall be designed to be functional, easily operable and economical, both for construction and operation.

6.1 Optimum Removal of Sediments

The settling basin shall be designed to remove as much of the sediment load in the water as is economically and hydraulically possible. As removal of all suspended sediments is physically not possibe, the design shall attempt to remove as much of the coarser fractions of the suspended load as possible so that the hydraulic transport capacity of the water conveyance system can be maintained and the sediment load to the turbines, valves, etc., is reduced to acceptable limits.

6.2 Efficient Flushing

The settling basin shall be designed to ensure efficient flushing of settled sediments so that frequent flushing during floods, when the sediment content of rivers is at its peak, is not required. The settling basin shall be planned and designed such that power generation is not interrupted, or reduced, during flushing operations.

7. TYPES OF SETTLING BASINS

One of the following types of settling basins, categorized in terms of their hydraulic functioning, shall be used for run-of-river hydropower projects:

a. Settling basin with periodic (intermittent) flushing.

b. Settling basin with continuous flushing.

7.1 Settling Basin with Periodic Flushing

Settling basins with periodic flushing shall be designed to operate in two distinct phases. In the first phase, the suspended sediments in the abstracted water shall be permitted to settle in the settling basin and water that is free from as much of the sediments as possible shall be conveyed to the water conveyance system. In the second phase, the deposited sediments shall be hydraulically removed from the settling basin through a flushing system using gravity flow of water at high velocities. During the flushing, water supply to the water conveyance system may either be stopped or alternately channeled through another settling chamber or bypass system.

7.2 Settling Basin with Continuous Flushing

Settling basins with continuous flushing shall be designed to supply sediment-free water to the water conveyance system through simultaneous settling of the suspended sediments and flushing of deposited sediments. The flushing shall be achieved by continuously abstracting water from the bottom of the settling basin during operation.

8. SELECTION OF TYPE OF SETTLING BASIN

The choice between settling basins with periodic or continuous flushing discussed in Section 7 shall be made based on the following factors:

a. Topography.

b. Availability of water.

c. Type and size of power plant.

d. Cost of construction.

e. Ease of operation and maintenance.

f. Power outage or reduction.

8.1 Topography

A settling basin with continuous flushing, which can function with a single chamber, may be preferred at narrow sites, provided the site is sufficiently long to accommodate the basin. At sites where sufficiently wide land with relatively plain topography is available, settling basins with periodic flushing may be considered.

8.2 Type and Size of Power Plant

The choice of settling basin shall be made based on whether the power plant is a high head or a low head plant.

8.3 Availability of Water

Settling basins with continuous flushing shall be preferred at headworks where the additional water needed for flushing is readily available. On the other hand, settling basins with periodic flushing may be adopted where additional water cannot be provided at the settling basin due to hydrological or other constraints.

8.4 Cost of Construction

The choice between the two types of settling basins shall be based on a comparison of the cost of the settling basin from perspectives of construction volume and complexity. The comparison shall consider the following factors:

a. Settling basins with continuous flushing generally require larger approach canals and settling basin chambers to accommodate the additional flow required for flushing. The additional construction thus necessitated may be offset to a certain extent by the lower requirements for dead storage for deposited sediments.

b. Settling basins with periodic flushing may require an increased volume of construction due to the greater dead storage needed for sediments settled between flushing operations and due to the need for multiple chambers for maintaining continuity of flow to the water conveyance system.

c. Hoppers used for settling basins with periodic flushing require complex construction. The use of sediment ejection pipes and valves with their flushing system also add to the cost of construction.

8.5 Ease of Operation and Maintenance

The settling basin shall be easily operable with minimal mechanical or human intervention for sediment flushing. This factor shall be particularly important to small and remotely-located projects where elaborate setups for operation and maintenance may not in place. For such projects, settling basins with continuous flushing may be the preferred option.

8.6 Power Outage or Reduction

The settling arrangement shall, as far as possible, ensure that the flushing of the settling basin does not result in stoppage or reduction of flow to the water conveyance system, resulting in power outage or reduction in power generation. A settling basin with continuous flushing may be the obvious choice to satisfy this requirement. Where this is not possible due to other constraints, a periodically flushing settling basin with multiple chambers shall be opted for.

9. TYPICAL COMPONENTS

Settling basins with periodic flushing shall consist of the following components (Figure 1):

a. An inlet transition at the entrance of the settling basin to distribute the flow uniformly over the cross-section of the settling basin chamber.

b. A regulator at the entrance of the chamber to control flow of water into the settling basin chamber.

c. A settling chamber to settle the sediments in the incoming water.

d. An emergency spillway to spill the settling basin design discharge into the parent river, if required

e. An exit regulator to control the flow of water from the settling chamber to the water conveyance system.

f. A flushing channel for flushing the settled sediment.

g. An exit transition to ensure smooth passage of desilted water from the settling basin chamber to the water conveyance system.

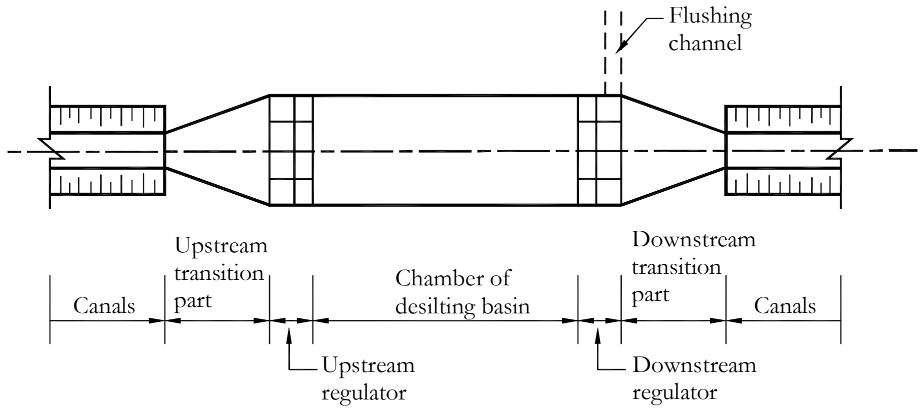

Figure 1: Typical components of settling basin

In addition to the above listed components, settling basins with continuous flushing shall have longitudinal channels in the settling chamber to facilitate continuous flushing of deposited sediments.

10. DESIGN OF INLET TRANSITION

The inlet transition for settling basins shall be designed to prevent turbulent flow at the entrance to the settling basin chamber. This shall be achieved through the design considerations discussed in the following sections.

10.1.1 Approach to Inlet

To maintain an even flow distribution at the start of the inlet transition, the approach canal to the settling basin shall have a straight alignment for a stretch equal to about ten times the width of the canal upstream of its junction with the transition. The hydraulic design of the canal shall eliminate secondary currents in its flow caused by rotational flow. It shall also ensure flow velocities in the range of 1.1 to 1.3 m/s.

10.1.2 Curved Approach Canal

If a straight approach canal prescribed in Section 10.1.1 is not possible, a skewed inlet may be provided to compensate for a skewed approach flow caused by a curved approach canal. The approach may result in the flow at the downstream end of the inlet transition being evenly distributed over its cross section for the design flow only.

To avoid secondary currents produced due to bends, an accelerated flow may be created downstream of the bend through a pressurized canal, and the flow may then be carefully retarded to achieve an even flow distribution. Hydraulic design of this arrangement shall be optimized through physical or numerical model tests. Due to the low transit velocities in the settling basin, the model tests shall be conducted at scales not less than 1:25.

10.1.3 Inlet Transition

The geometry of the inlet transition shall ensure that the flow is evenly distributed over the width and the depth of the settling basin, especially in the desanding chamber, for all ranges of flow through the settling basin. For this purpose, a symmetrical and smooth layout of the inlet expansion shall be designed to prevent the flow from separating from the sidewalls and bottom of the transition. This shall be achieved by providing an opening angle of the inlet transition in the range of 7° to 10° along with smooth curvature (Figure 2).

Where space constraints do not permit the long transition resulting from the recommended opening angles, the inlet transition may be shortened through guide walls in the transition (Figure 2) so that the opening angle between two guide walls is small enough to prevent separation. The downstream end of the guide walls shall be carefully shaped so that the water velocity at the transition outlet remains parallel to the longitudinal axis of the settling basin and does not point towards the sidewalls.

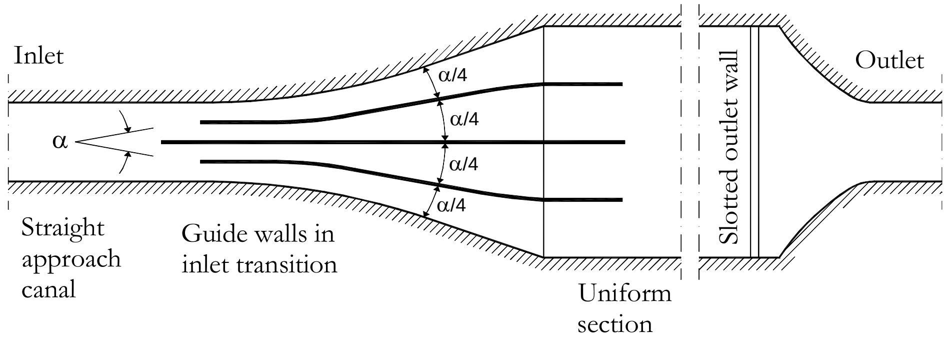

Figure 2: Inlet transition with guide walls

10.1.4 Tranquilizer / Baffle Blocks

If space constraints at the headworks site do not permit a straight section of approach canal or conduit to the settling basin, even flow in the settling basin may be achieved using a tranquilizer or baffle blocks. The tranquilizer shall be designed to force the incoming water flow to be distributed across the settling basin cross-section. Care shall be taken to minimize head losses resulting from the use of tranquilizers. Suitable measures shall also be adopted for preventing trash, floating bodies and gravel from clogging parts of the tranquilizer. For major structures, baffle blocks shall be designed based on model tests.

11. DESIGN OF SETTLING BASIN

Design of settling basin shall include hydraulic design of its chamber and flushing systems. It shall also cover the structural design of these components.

11.1 Hydraulic Design

The hydraulic design of the settling basin shall consist of determination of its size and shape to secure:

a. An even flow distribution between parallel settling basins for various flows.

b. An even flow distribution internally inside each basin for various flows.

c. Efficient removal of deposits during flushing of the basin.

The design shall be based on a careful selection of sediment particle size and percentage to be settled, considering plant maintenance.

11.1.1 Design Data

Hydraulic design of the settling basin shall be performed based on sediment and design data, design parameters and standard data on settling velocity, sediment concentration and transport capacity of suspended sediments.

11.1.1.1 Sediment Data

Sediment data required for the design shall include the following:

a. Sediment concentration of the river flow, characterized in terms of the following:

   i. Mass content of suspended sediments, ρ, per unit volume of water.

   ii. Concentration of suspended sediments, μ, expressed in litres of suspended sediments per unit volume of flowing water (or ppm).

b. Particle size distribution of sediments.

c. Mineralogical composition of sediments.

These data shall be obtained through procedures discussed in Part 1C of the guidelines.

11.1.1.2 Project-related Data

Project-related data needed for design of settling basins shall consist of the following:

a. Sediment size and percentage to be settled.
b. Design discharge and flushing discharge.
c. Type of plant and its hydro-mechanical equipment.

11.1.1.3 Design Parameters

The parameters needed for design of the settling basin shall include the following:

a. Mean depth of flow
b. Mean longitudinal velocity
c. Flushing discharge
d. Flow velocity during flushing or sediment concentration of flow functionally related to this velocity

The values of these parameters shall be selected such that the resulting size and shape of the settling basin yield an economical and practical design.

11.1.1.4 Settling Velocity

For design, the settling velocity, ω, of particles with diameter up to 1.5 mm may be adopted from Table 1. Likewise, the velocities for particles larger than 1.5 mm may be adopted from Table 2.

Table 1: Settling velocity of particle size up to 1.5 mm for different water temperatures

Particle size (mm)	Settling velocity ω (mm/s) for different water temperatures				
	10º C	15º C	20º C	25º C	30º C
0.001	0.0007	0.0008	0.0009	0.001	0.0011
0.010	0.0680	0.0790	0.0900	0.100	0.1100
0.020	0.2740	0.3160	0.3600	0.400	0.4500
0.030	0.6180	0.7100	0.8100	0.900	1.0120
0.050	1.7170	1.9730	2.2700	2.500	2.8120
0.070	2.5100	2.8800	3.2500	3.650	4.1000
0.100	5.1200	5.8800	6.6300	7.440	8.3700
0.200	17.1100	18.7600	20.4200	22.060	23.720
0.300	28.3100	29.9600	31.6200	33.260	34.920
0.500	50.7100	52.3600	54.0200	55.660	57.320
1.000	106.7100	108.3600	110.0200	111.660	113.320
1.500	162.7100	164.3600	166.0200	167.660	169.320

(Source: Zhurablov, 1975)

Table 2: Settling velocity of sediment particles with size larger than 1.5 mm

d (mm)	ω (mm/s)	d (mm)	ω (mm/s)	d (mm)	ω (mm/s)	d (mm)	ω (mm/s)
1.50	170.0	4.0	268.5	9.0	403.0	20.0	602.0
1.75	178.0	5.0	300.0	10.0	425.0	22.5	637.0
2.00	190.0	6.0	329.0	12.5	477.0	25.0	672.0
2.50	212.5	7.0	355.0	15.0	520.0	27.5	706.0
3.00	232.5	8.0	380.0	17.5	562.0	30.0	736.0

(Source: Zhurablov, 1975)

11.1.1.5 Lifting (Buoyant) Velocity of Flow

This lifting velocity of flow shall be determined through suitable empirical formulae. The following formula may be used for this purpose (Zhurablov, 1975):

Eq. 1

$$u_l = 0.065(v - 0.05)\frac{\sqrt{n\,v}}{\sqrt[3]{R}}$$

where u_l is the lifting velocity of flow, n is the roughness coefficient of the bed material, v is the longitudinal flow velocity and R is the hydraulic mean radius.

11.1.2 Design Considerations

The following factors shall be considered in the design of the settling basin and its flushing system:

a. Suspended sediments consist of fine particles of hard or soft rock or soil with different diameters. Most Nepali rivers carry sediments of hard rock origin like quartzite, granite, senite, basalt, gneiss, etc.
b. Concentration of sediments changes frequently with the river flow.
c. Required removal percentage of suspended sediments depends on the quality of material (hard or soft), type of turbine and available head for power generation.

11.1.3 Design Criteria

The design of the settling basin shall be based on the following criteria:

a. The settling basin shall be long enough, but economically effective, to allow settling of sediments of specified size depending upon the origin of sediments.
b. The length of the settling basin shall be determined mainly by the settling velocity of sediment to be eliminated and by the longitudinal flow velocity.

11.1.4 Design Assumptions

The design of the settling basin shall be based on the following assumptions:

a. The sediment concentration of flow entering the settling basin is equal to the design sediment concentration of the river flow.
b. Supply of the design flow to the power canal takes place with simultaneous settling and flushing of sediments or with sediment settling followed by intermittent flushing.
c. Water level in the basin is horizontal.
d. In continuous flushing, the depth of flow in the basin is constant, but the flow regime is non-uniform due to change in flow discharge along the basin length during sediment flushing.
e. Average velocity of flow in the settling basin is constant and does not change in time or space.

f. The settling velocity is not affected by the temperature of water.

g. The vertical distribution of sediments may be triangular, rectangular or trapezoidal.

h. Flushing of settled sediments takes place in uniform regime of flow.

11.1.5 Settling Basin with Periodic Flushing

11.1.5.1 General Arrangement

The settling basin with periodic flushing shall normally consist of two or more chambers. These chambers shall be separated from each other by longitudinal divide walls (Figure 3). These walls shall have a rectangular or trapezoidal cross-section, and their top level shall be fixed providing a freeboard of 0.3 to 0.5 m above the full supply level, considering the mode of plant operation.

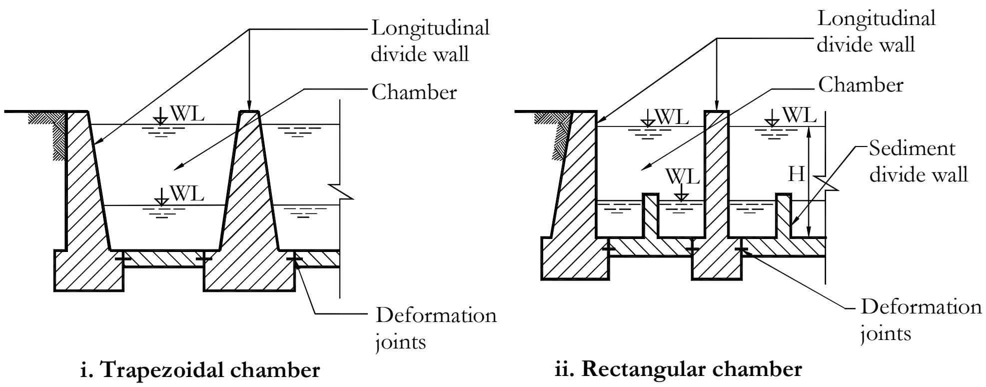

Figure 3: Cross-section of settling basin with periodic flushing

The flushing arrangement for the settling basins with periodic flushing shall include a flushing regulator, a flushing gallery and a flushing channel as an extension of the flushing gallery (Figure 4).

The flushing gallery shall be provided at the end of the settling basin chamber across its entire width. The bed of this gallery shall lie below the flushing crest.

Deformation joints along the length of the settling basin may be provided in concrete and reinforced concrete structures in accordance with standard practice. These joints shall be sealed with waterstops.

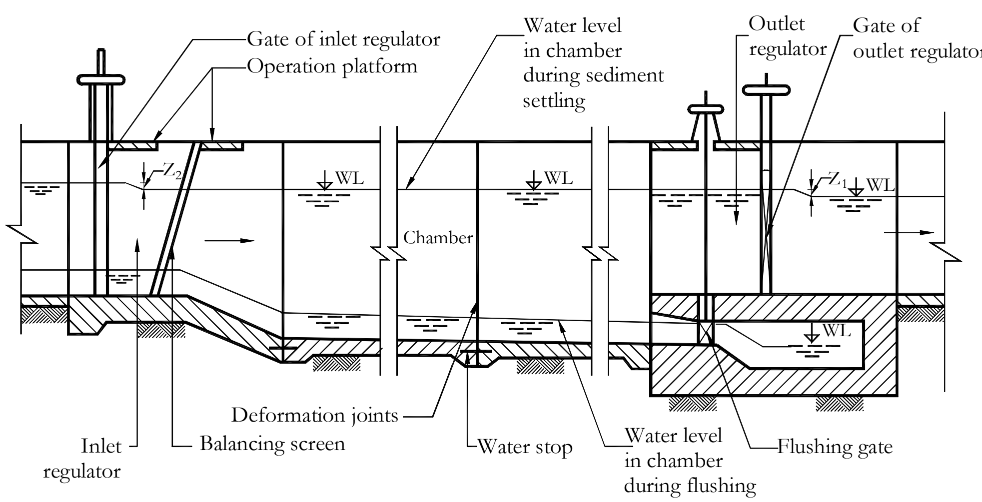

Figure 4: Longitudinal section of settling basin with periodic flushing

11.1.6 Settling Basin Design

The settling basin chamber shall be designed for the discharge to be passed by the settling basin to the power canal. Its cross-sectional area shall be calculated as (Zhurablov, 1975)

Eq. 2 $$A_{d\cdot b} = \frac{Q_C}{V_m}$$

where $A_{d.b}$ is the flow area, Q_c is the discharge to be passed to the power canal and V_m is adopted mean flow velocity during sediment settling. For a rectangular section, the chamber width for given mean flow depth will be

Eq. 3 $$B_{d\cdot b} = \frac{A_{d\cdot b}}{H_m}$$

where $B_{d.b}$ is the width of the chamber and H_m is the adopted mean flow depth.

For a single chamber settling basin, the length of the chamber shall be determined as

Eq. 4 $$L_{cb} = kH_m \frac{V_m}{\omega - u_*}$$

where k is a safety factor taken equal to 1.2 to 1.4, ω is the settling velocity of the sediment to be deposited on the chamber bed and u_* is the shear velocity (lifting velocity) determined from the equation

Eq. 5 $$u_* = \frac{0.042\, v_m}{R^{1/6}}$$

in which R is hydraulic mean radius of the flow section.

For given flushing velocity, the flow depth of chamber during flushing shall be equal to

Eq. 6 $$h_f = \frac{Q_f}{B_{cb} \cdot V_f}$$

where h_f is the flow depth of chamber during flushing, Q_f is the adopted flushing discharge and V_f is the flushing velocity. For computed h_f,, the bed slope of chamber, S_{cb}, shall be determined using Manning's equation:

Eq. 7 $$S_{cb} = \frac{n^2 V^2}{R^{4/3}}$$

In the absence of sediments, the flow depth at the beginning of chamber shall be given by

Eq. 8 $$H_1 = H_m - S_{cb} \frac{L_{cb}}{2}$$

and at it end by

Eq. 9 $$H_2 = H_m + S_{cb} \frac{L_{cb}}{2}$$

The chamber bed slope shall be determined for different of values of V_f, and one of the computed slopes shall be adopted based on its appropriateness.

The total volume of sediments deposited in the chamber with bed slope in the direction of flow shall be taken to be 0.6 V_{dead}, where V_{dead} is dead volume of chamber of the settling basin. For design purposes, the chamber bed shall be taken to be horizontal with flow depth H_m, and the total volume of the deposited sediments in the chamber at the time of its flushing shall be taken as

Eq. 10
$$V = V_1 + V_2$$

where V_1 is the volume of sediments formed as the result of deposition of given particle sizes and larger and V_2 is the volume of sediments formed as the result of deposition of the particles lesser than that of given sizes. The value of V_1 shall be obtained from the equation

Eq. 11
$$V_1 = 0.001\,\mu \cdot Q \cdot t$$

where μ is the volumetric sediment concentration of given particle size and larger, litre/m^3, t is the settling duration of sediments in s and Q is the flow discharge passing through the chamber, m^3/s. Likewise, the value of V_1 shall be obtained from the equation

Eq. 12
$$V_2 = 0.001\frac{Q \cdot t}{H_m}\left(\mu_1 h_1 + \mu_2 h_2 + ... + \mu_n h_n\right)$$

where $\mu_1, \mu_2 ... \mu_n$ is the volumetric sediment concentration of separated particles of the suspended sediments in litre/m^3, H_m is the mean flow depth in the chamber with real distribution of sediments in m and h_1, $h_2 \ldots h_n$ are the flow depths from which the separated particles of suspended sediments are settled down in the chamber in m. The magnitudes of the flow depths shall be found by the approximate relationship (Zhurabov, 1975)

Eq. 13
$$h_n = L_{ch}\frac{\omega}{V'_m}$$

where

Eq. 14
$$V'_m = \frac{q}{H_m - 0.5\,\Delta H}$$

with

Eq. 15
$$\Delta H = H_m - h_c$$

From Eq. 10, the sedimentation time of chamber shall be determined for the known geometrical dimensions of chamber and provided sediment concentration of the river flow, taking V = 0.6 V_{dead}. In these computations, particles less than 0.05 mm may be neglected as all of these particles practically pass with flow into the power canal.

11.1.6.1 Design for Sediment Flushing

Design of chamber flushing of the settling basin shall involve determination of the transporting capacity of the flow for sediment flushing and the time of flushing.

Flushing Capacity

The flushing capacity shall be determined as (Zhurabov, 1975)

Eq. 16
$$\rho_{tr} = \frac{\left(V_f - 0.35\right)^3}{b_f^2}$$

where ρ_{tr} is the mass of sediments transported in kg, V_f is the flushing velocity in m/s and h_f is the flow depth during flushing in m. Eq. 16 may also be used to determine the flushing velocity if the transporting capacity of the flow, ρ_{tr}, is known, and then the flushing gallery bed slope of the settling basin may be obtained from Eq. 7.

The flushing duration of the settling chamber shall be determined by the expression

Eq. 17
$$t_f = \frac{16.7\gamma_s \cdot 0.6V_{dead}}{\left(\rho_{tr} - \rho_0\right)Q_f}$$

where t_f is the flushing time in minutes, γ_S is the unit weight of sediment to be flushed and ρ_0 is the sediment concentration of the flow entering the chamber during flushing, usually taken to be the sediment concentration of river flow.

11.1.7 Settling Basin with Continuous Flushing

11.1.7.1 General Arrangement

Settling basins with continuous flushing may have one or more chambers (Figure 5). More than one chamber may be desirable to enable dewatering of one basin during the dry season for inspection and maintenance of the flushing system, etc. without affecting the operation of the plant. Alternately, single chamber settling basin with a bypass channel for this purpose may be provided.

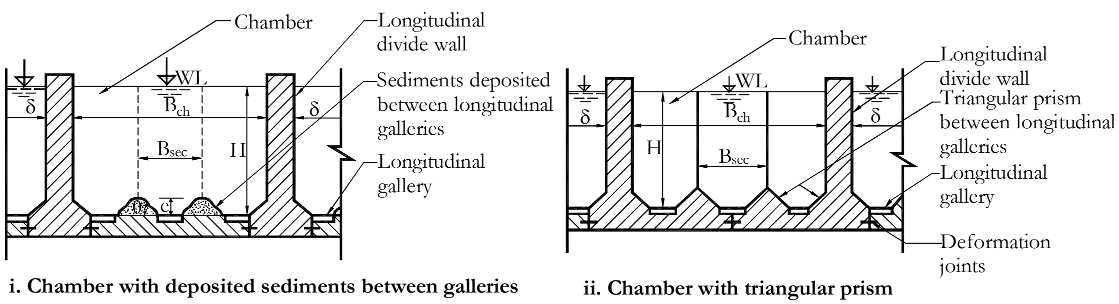

Figure 5: Cross section of settling basin with continuous flushing

This settling basin shall be provided with a series of bottom galleries along the length of its chamber to pass settled sediments (Figure 6). The flow area of these galleries shall be increased along the length to accommodate the increasing flushing discharge. The channels shall be covered with perforated screens kept horizontally and flush with the chamber bed.

A flushing gallery shall be provided at the end of the settling basin chamber. The crest of the flushing gallery shall be placed at a depressed level to create unsubmerged flow regime in it.

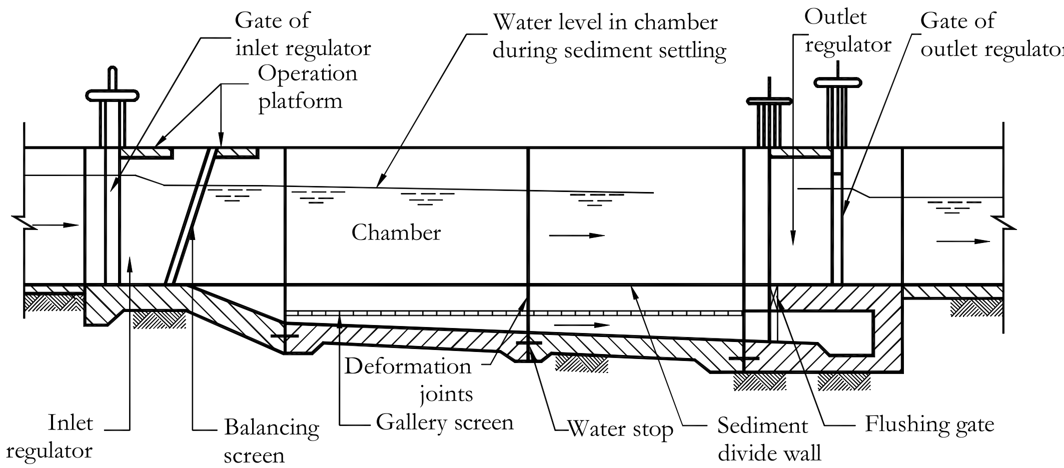

Figure 6: Longitudinal section of settling basin with continuous flushing (Zhurablov, 1975)

11.1.7.2 Design Parameters

Settling basins with continuous flushing shall be designed for the discharge of the power canal, Q_c, sediment concentration of the flow entering the settling basin, ρ_0, and the sediment fraction sizes to be settled in the basin. Other design parameters may be chosen from the following:

a. Mean flow velocity in the chamber, V_m = 0.2 to 0.3 m/s.

b. Flow depth in the chamber, H = 1.0 to 15.0 m.

c. Flushing discharge through bottom (flushing) gallery, Q_f = 10 to 25 percent of Q_c.

d. Flushing velocity in the gallery.

11.1.7.3 Design of Settling Chamber

Design of the settling chamber shall include determination of its geometrical dimensions, quantity of sediments to be deposited in the chamber, fixation of sediment concentration of the flow passing through the canal and flushing gallery and computation of dimensions of the flushing channel for sediment transport.

The flow area of the chamber, A, shall be determined by the formula (Zhurablov, 1975):

Eq. 18 $$A = \frac{Q_c + 0.5Q_f}{V_m}$$

The number of sections in the basin shall be obtained as

Eq. 19 $$n = \frac{A}{a}$$

where a is the flow area of the section. The value of n obtained thus shall be rounded off to a whole number, and the flow area of the settling basin and its flow velocity shall be accordingly corrected.

The length of the settling basin, L, shall be determined by the formula

Eq. 20 $$L = H\frac{V_m}{\omega} - \frac{Q_f}{B_m(\omega - u_*)}$$

where ω is the settling velocity of the given sediment particles to be settled in the settling basin and B_m is the mean width of the settling basin, given by

Eq. 21 $$B_m = \frac{A}{H}$$

11.1.7.4 Design of Sediment Settling

Deposition of the sediments in the basin shall be calculated for one section, and the values for all other sections shall be taken in proportion to their width. For this purpose, the design sediment and larger fractions shall be eliminated from consideration, and the quantity of remaining sediment fractions with small intervals is adopted for 100%.

The depth of flow, h, with which sediment particles less than fixed size settle down partially in the basin shall be found from the formula (Zhurablov, 1975)

Eq. 22 $$h = L\frac{\omega}{V_m} + \frac{Q_f}{B_m V_m}$$

For each diameter of sediment particles less than fixed, the percent of sediment deposition in the basin shall be calculated by the expression

Eq. 23 $$p' = p\frac{h}{H_m}$$

where p' is the percent of sediment deposition in the basin, p is the percentage of the considerable particle diameters of the suspended sediments and H_m is the mean flow depth given by

q. 24 $$H_m = \frac{A}{B}$$

At the end of calculations, the weight of sediment for each particle size shall be determined for its deposition in the settling basin and passage in the canal, summing all particle sizes.

11.1.7.5 Design of Sediment Flushing

The sediments to be settled down in the settling basin shall be equal to (Zhurablov, 1975)

Eq. 25 $$\rho_b = \rho_1 + \rho_2$$

where ρ_1 is the sediments of the fixed fractions and larger and ρ_2 is the sediments lesser than fixed fractions.

The discharge entering the settling basin shall be

Eq. 26 $$Q_b = Q_c + Q_f$$

At the same time, sediments with flow entering the basin, R, shall be

Eq. 27 $$R = \rho_0 Q_b$$

Assuming that there is no loss of water in the basin, the amount of sediments, T, passing into the power canal shall be

Eq. 28 $$T = \rho_c Q_c$$

where ρ_c is sediment concentration of flow entering the canal whose value shall be known from calculation of sediment deposition.

In the settling basin, the amount of sediments settled down shall be

Eq. 29 $$Z = R - T$$

For adopted flushing discharge of flow Q_f, the sediment concentration of flow to be flushed at the end of the bottom gallery shall then be

Eq. 30 $$\rho_{gal} = \frac{Z}{Q_f}$$

11.1.8 Design of Bottom Gallery

The bottom gallery shall have a pressure regime of flow with variable discharge increasing to the end of section of the settling basin. Since determination of flow parameters for such regime is difficult, approximate methods shall be used for this purpose. The bottom gallery along its whole length shall be divided into parts within which the flow shall be considered to be uniform with design parameters that are taken at the middle of each part. On the basis of computation, the dimensions of the gallery, losses of head along its length and location of the piezometric line shall be determined.

11.1.9 Design of Flushing Gallery

Flushing channel below the crest of exit regulator on the canal may be designed for pressure or non-pressure regime. For pressure regime, the difference of head between the peizometric line in the settling basin and water level in the river (at the outlet) shall be known. For non-pressure regime, this channel shall be designed as an open channel with uniform flow similar to the settling basin with periodic flushing.

11.1.10 Sediment Trapping Efficiency

The sediment trapping efficiency of the settling basin is considered to be quite essential for designing this basin. In an ideal basin by means of a basin without any turbulence, all the suspended sediment particles with the settling velocity equal to or greater than w will be trapped:

Eq. 31 $$w = \frac{Hv_m}{L} = \frac{Q}{A_s}$$

where H is the depth of flow in the basin, v_m is the mean horizontal velocity of flow, L is the length of the basin, Q is the discharge entering the basin and A_s is the net water surface area of the basin.

As there is turbulence in the low, some sediment particles will not settle as fast as the settling velocity prevails because the turbulence will always move some particles in the upward direction. Camp's diagram () considers the effect of turbulence theoretically on the sediment trapping efficiency of the settling basin.

The sediment trapping efficiency, η, based on the sediment particle approach for design of the settling basin is found from Camp's diagram, considering the dimensionless parameters w/u_* and $(wA_s)/Q$, where w is the settling velocity of sediment particles and u_* is the shear velocity that can be found theoretically using Manning's formula

Eq. 32 $$u_* = \sqrt{gRS_e}$$

where g is the acceleration due to gravity, R is the hydraulic mean depth of flow, n is the Manning's rugosity coefficient, A is the cross-sectional area of flow of the basin and S_e is the energy gradient given by

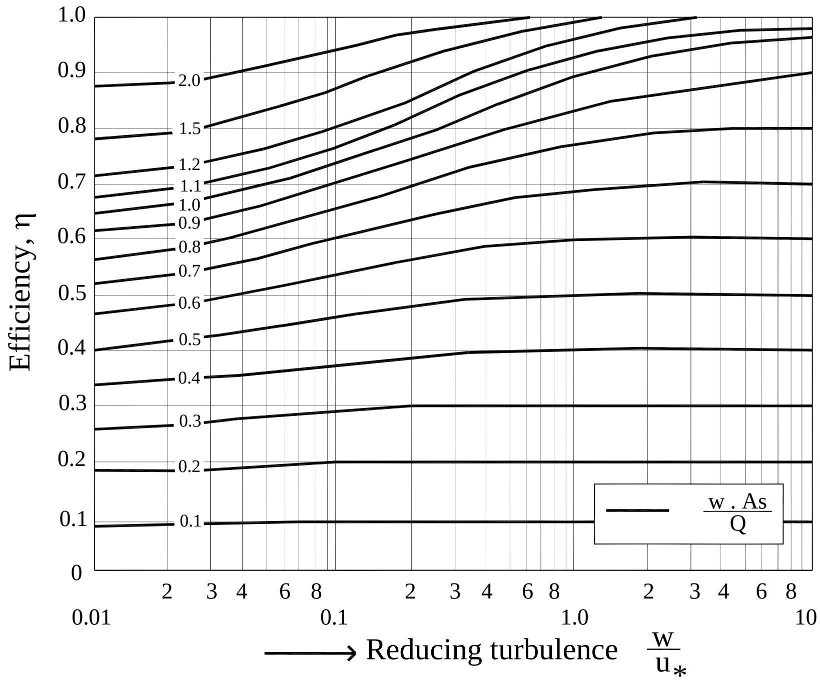

Figure 7: Camp's diagram for sediment trapping efficiency

Eq. 33
$$S_e = \left(\frac{Q}{nAR^{2/3}} \right)^{1/2}$$

For the practical case, the shear velocity shall be determined from the following formula instead of from

Eq. 34
$$u_* = \frac{0.042 v_m}{R^{1/6}}$$

Vetter's method for determining the sediment trapping efficiency, based on the sediment concentration approach and simplified version of Hazen's method, is expressed by the formula:

Eq. 35
$$\eta = 1 - e^{-\left(\frac{wA_s}{Q}\right)}$$

This formula does not consider the effect of turbulence in flow of the basin on the sediment trapping efficiency in it. As such, this formula may be used with care.

11.1.11 Flushing Gates

Flushing gates shall be placed at the end of the settling basin. They shall be provided with easy access for operation.

11.2 Structural Design

The structural design of the settling basin shall be based on the design of an open water tank. The loads listed below shall be considered in the design:

a. Dead loads.

b. Hydrostatic loads.

c. Uplift pressures.

d. Silt load.
e. Earth pressures.
f. Earthquake loads.

11.2.1 Design of Outer Walls

The outer wall shall essentially act as retaining walls designed for following load conditions listed in Table 3.

Table 3: Load conditions for design of settling basin walls

Condition	Description
Usual	Normal earth pressure behind the wall Sudden drawdown
Extreme	Usual conditions with earthquake

11.2.2 Design of Divide Walls

The divide wall between the settling basin chambers shall be designed for the worst combination of design loads. For this purpose, the load conditions listed in Table 4 shall be considered.

Table 4: Load conditions for design of divide walls

Condition	Description
Usual	Design water level in one basin Design silt level in the same basin Adjoining basin empty
Extreme	Usual conditions with earthquake

11.2.3 Design of Chambers and Hoppers

For optimal design, the settling basin chambers and hoppers shall be analyzed and designed as folded plate structures. The analysis shall be conducted for the most severe condition in which the chamber or hoppers are empty and are subject to dead loads, lateral soil pressure and upward bearing pressure.

11.2.4 Lining

The bed and wall surfaces of the settling basin and flushing galleries shall be lined with abrasion-resistant material. Based on site conditions, the material may be chosen from the options listed in Section 11.2.7 of Part 2B of the guidelines

12. DESIGN OF OUTLET TRANSITION

Depending on the alignment of the power canal intake, the outlet transition for the settling basin may be straight or curved in plan. In either case, the contraction part of the transition shall be symmetrical and smooth to prevent the flow separation from its side walls and bottom. This shall be achieved by providing a closing angle of the outlet transition in the range of 10° to 15°.

The invert level of the gate sill for the power canal intake shall be located at an elevation slightly higher than the invert level of the outlet transition. Transitions or piers at this location shall be semi-circular or elliptical in shape.

13. DESIGN OF EMERGENCY SPILLWAY

The emergency spillway shall an integral part of the settling basin. It shall be designed to safely discharge the entire design flow of water coming from an approach canal or an intake back into the parent river. It shall also pass floating materials present in the settling basin to the river.

13.1 General Arrangement

The emergency spillway shall consist of a chute or an ogee type overflow structure. A stilling basin shall be provided at the end of the spillway to dissipate surplus energy in the flow. This flow shall be returned to the river directly or through an outlet canal. In rare cases, the emergency spillway may be combined with the bottom sediment flushing outlet on either side of the abutment wall.

The emergency spillway may be located anywhere along the side-wall of a settling basin on the riverside. However, its preferred location shall be the downstream end of the settling basin.

The crest of the emergency spillway shall be aligned perpendicular to the longitudinal flow axis of the settling basin in order to increase its discharging capacity. The crest level shall usually be fixed at the full supply level in the settling basin, which also helps to pass the floating material downstream of it.

13.2 Flow Regimes

The flow on the emergency spillway shall be non-pressured. Beyond the spillway crest, the non-pressure flow shall be super critical with high turbulence towards the end of the stilling basin. After dissipation of surplus energy in the stilling basin, non-pressure flow shall occur along the entire length of the outlet canal in sub-critical, critical, or super critical flow regime depending upon the topographical condition of the headworks site.

If the emergency spillway is combined with the bottom sediment flushing outlet, the flow regime on that spillway bay shall take a form of the gravel-water mixture with sediment after the toe of that spillway. For this case, the sediment transporting capacity of the outlet canal shall be checked to ascertain its ability to flush all the sediments coming from the settling basin into the parent river, where the end outlet structure shall be designed strongly.

13.3 Hydraulic Design

Hydraulic design of the emergency spillway shall involve determination of the crest length of the spillway, profile of the spillway glacis, length and elevation of the stilling basin, dimensions of the outlet canal and the outlet end structures.

The spillway crest and profile shall be designed based on provisions for design of overflow sections of diversion structures discussed in Part 2B of the guidelines. However, as the design discharge of the intake is always significantly less than that for the overflow section of the diversion structure and the height of the emergency spillway is low, the design criteria for the emergency spillway shall not be as stringent as those for the overflow section.

The stilling basin for the emergency spillway shall be designed according to the provisions of Part 2C of the guidelines. The outlet canal shall be designed as an open flume using Manning's or Chezy's formula as described for approach canals in Part 2F of the guidelines.

14. PHYSICAL AND NUMERICAL MODELING

The hydraulic design of settling basins and their sediment flushing system shall be confirmed with hydraulic model test. A mathematical simulation of the design, based on computational fluid dynamics, may also be performed to finalize the size of the settling basin and the sediment flushing system. Such simulation shall include the effect of inflow and outflow conditions in the computation of the trap efficiency.

PART 2I – GATES

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2I

Gates

1. PURPOSE

The purpose of this section of the *Design Guidelines for Headworks of Hydropower Projects* is to provide technical criteria and guidance for the planning and design of Gates for run-of-river hydropower projects.

2. SCOPE

The guidelines presented in this document covers choices of gates and design of vertical and radial gates. The guidelines are general in nature and they were developed in accordance with standards and guidelines prevailing in different countries with due consideration to Nepali conditions and experience.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Gate or Shutter	A closure device in which a leaf or a closure member is moved across the waterway from an external position to control the flow of water.
Bulkhead Gate	A gate operated only under balanced pressures and invariably kept in either fully-open or fully-closed position.
Anchorage	A structural member embedded in concrete for aligning and holding the embedded parts of a gate in position.
Deep Seated Radial Gates	Low-level radial outlet gates.
Flushing Gate	A gate located at various locations of a hydropower plant to flush out accumulated sediment.
Emergency Gate	A gate provided on the upstream of a service or a regulating gate to shut off the flow under unbalanced head.
Fixed Wheel Gate	A gate mounted with wheels having axles fixed to the gates.

Flap Gate	A gate which operates by rotation about a hinge or hinges.
Intake Gate	A gate located at the upstream end of a river outlet, conduit or penstock.
Multileaf Gate	Hook gate with vertical gate or combination of flap gate with radial gate, etc.
Radial Gate	A hinged gate, the leaf of which is usually a circular arc with the centre of curvature at the hinge or trunnion.
Stoplog	A log, plank cut timber, steel or concrete beam fitting into end grooves between walls or piers to close an opening under unbalanced conditions.
Vertical Gate	A gate operating in vertical grooves.

Bottom Seal	A seal provided at the bottom of the gate leaf.
Gate Leaf	The main body of a gate consisting of skin plate, stiffeners, horizontal girders and end girders.
Gate Seal	A device for preventing the leakage of water around the periphery of a gate.
Gate Sill	The top of an embedded structural member on which a gate rests in closed position.
Seal Plate	A metal plate mounted on a gate leaf to transfer water pressure to the seat and to act as a seal.
Top Seal	A seal provided at the top of a gate leaf or gate frame.
Trunnion Axis	The axis about which a radial gate rotates.

4. DESIGN PHILOSOPHY

4.1 Objectives

a. The gates shall be designed for the hydrostatic and hydrodynamic forces taking into consideration forces arising from wave effects, seismic loads and ice formation wherever applicable.

b. The additional water head to the static head to account for the sub-atmospheric pressure downstream of gates located in conduits/sluices should be specified to the designer.

c. The gate is normally designed to close under its own weight with or without addition of ballast but sometimes it may require a positive thrust for closing, in which case hoist shall be suitable for that purpose.

4.2 Design Criteria

The gate, in general, shall satisfy the following criteria:

a. It shall be reasonably watertight, the maximum permissible leakage being not more than 5 litres/min/m length of seal in case of crest gates and medium head conduit gates. The figure of permissible leakage is the upper limit before which remedial measures shall be required to rectify defects;

b. It shall be capable of being raised or lowered by the hoist at the specified speed;

c. Power operated gates shall normally be capable of operation by alternate means in case of power supply failure;

d. If meant for regulation, it shall be capable of being held in position within the range of travel to pass the required discharge without cavitations and undue vibration; and

e. Whenever necessary, model studies may be carried out for high head regulating gates.

The crest gate should generally be self-closing. The closing moment provided by the moving parts of the gate in any position should always be greater than the forces opposing the closure movement of the gate whether it is with the top seal against a breast wall or without top seal. However, for conduit gates, it may become necessary to provide a positive thrust for closing, in which case the hoist provided shall be suitable for the purpose.

4.3 Occasional Forces

4.3.1 Earthquake Effect

Earthquake forces shall be computed in accordance with criteria for earthquake resistant design and the gate designed accordingly.

4.3.2 Wave Effect

For very wide and big reservoirs, the effect of wave height due to storms, etc. in causing increased loading on the gate, shall also be considered.

4.3.3 Occasional Overtopping

Whenever occasional overtopping of gate is allowed the design of gate shall be checked for increased stresses. Proper provision shall be made to protect gate during occasional overtopping.

4.3.4 Permissible Stresses

The allowable stresses in the various parts of the gate under the action of occasional forces shall be increased by 33 percent of the permissible stresses subject to the maximum of 85 percent of the yield stress. In case of nuts and bolts, increase in stress shall not be more than 25 percent of allowable stress (Refer IS 4622:2003). The permissible values in welded connections shall be the same as permitted for parent material.

The earthquake forces, the wave effect and occasional overtopping shall not be considered to act simultaneously while computing the increased stress in the gate.

4.4 Ice Load

4.4.1 Ice Impact and Ice Pressure

Provided local conditions do not impose other values, ice impact and ice pressure shall be taken into account in such a way that the water pressure triangle shall be replaced as given below.

i. In waters with ice thickness greater than 300 mm, by an even surface pressure of 30000 N/mm^2 up to 3 m depth; and.
ii. In waters with ice thickness up to 300 mm by an even surface pressure of 20000 N/mm^2 to 2 m depth.

4.5 CORROSION CONTROL

Corrosion damage will occur over time and can impair structural and operational capacity of gates. To minimize future structural problems and high maintenance and rehabilitation costs, resistance to corrosion shall be considered in the design process. Gates are subject primarily to localized corrosion (i.e., crevice corrosion or pitting corrosion), general atmospheric corrosion, or mechanically assisted corrosion. Prudent design and maintenance practices can minimize these types of corrosion. Corrosion of gates is best controlled by application of protective coatings, but is also effectively controlled by proper selection of materials, cathodic protection, and proper design of details. The selection of corrosion protection alternatives depends on the environment in which the gate will function.

4.5.1 Coating Systems

Application of coating systems is the primary method of corrosion protection for gates. Coating systems include alkyd enamel, vinyl, and epoxy paint systems. Thermal spraying (metallizing) should be considered when extreme abrasion is predicted or when the use of volatile organic compounds is restricted. Acceptable standards shall be followed for selection, application and specifications of coating systems and thermal spraying.

4.5.2 Cathodic Protection

Cathodic protection shall be used in the more corrosive environments to supplement the paint coatings. Since corrosion is a continuing process of removing electrons from the steel, cathodic protection introduces a low current to counteract this effect. This essentially causes all parts of the structure to be cathodic. Cathodic protection is achieved by applying a direct current to the structure from some outside source. The direct current can be invoked either by impressed current or sacrificial anodes attached to the gate. Sacrificial magnesium anodes are often installed on gates used in fresh water when carbon and stainless steels are in contact with each other. For example, anodes would be used when a painted mild steel gate is in contact with stainless steel tracks and rollers and also in contact with the stainless steel guides through the bearing shoes, rollers, or wheels. Unfavorable area differences such as a small anode (less noble mild steel 99 percent covered with paint) and a cathode many times larger (more noble stainless steel bare) will cause rapid development of pits at imperfections in the paint coating. Magnesium, being less noble than either mild steel or stainless steel, becomes a sacrificial anode and will protect these flaws in the paint coating and also protect oxygen-deficient areas on the stainless steel. To provide adequate protection, anodes shall be within 600 mm and in line of sight of the surface they are protecting.

4.5.3 Control Contamination

Metallic contamination of the metal surface can cause galvanic corrosion. Nonmetallic contamination on stainless steel can result in loss of passivity at the contamination sites or create oxygen concentration cells, which can cause pitting. Such components as stainless steel rollers, wheels, axles, track plates, seal plates, and guides shall be passivated after fabrication with a nitric acid solution. During manufacturing, metals may acquire contamination from metal forming and machining operations. Avoidance of contamination, or the discovery and removal of prior contamination on metals, is critical at the construction site during erection or installation of the structure or equipment.

4.5.4 Design Consideration for Corrosion Prevention

Structural detailing has a significant impact on corrosion prevention. Structures shall be detailed to avoid conditions that contribute to corrosion. The following items shall be considered in the design process:

a. Structural members shall be detailed such that all exposed portions of the structure can be properly painted or coated.

b. Provide drain holes to prevent entrapment of water. Locate extra large drain holes in areas where silt or sand may be trapped.

c. Avoid lap joints, but where used, seal weld the joint so that water cannot be trapped between the connected plates.

d. Grind slag, weld splatter, or any other deposits off the steel. These are areas that form crevices that can trap water.

e. Where dissimilar metals are in contact (generally carbon steel with either stainless steel or bronze), provide an electric insulator between the two metals and avoid large ratios of cathode (stainless steel) to anode (carbon steel) area.

f. Use continuous welds in lieu of bolts where possible with caution given to the effect of or susceptibility to fracture.

g. Break or grind sharp corners or edges to a minimum 1-mm radius to allow paint or coating to properly cover the surface.

h. Avoid designs with enclosed spaces. If such spaces cannot be avoided, make them large enough for maintenance work and painting, or provide cathodic protection. In some gates it may be possible to fill and seal the space with a non-corrosive liquid or solid. This technique has been used on tanks for floating fish entrance gates.

i. Consider using corrosion-resistant metal for areas that will be inaccessible for replacement.

j. If anodes are used, allow enough room for maintenance workers to replace them.

4.5.5 Safety

Corrosion may be the cause of catastrophic damage or loss of life due to failure of a gate or structural members of a gate. Particular attention shall be applied to structural members that are inaccessible to inspection or accessible only for infrequent inspection. Prevention of corrosion failures shall be investigated during the design of the gate. Where corrosion failure will place human life at risk, the most current methods of corrosion control shall be employed.

5. TYPES OF GATES AND HOISTING ARRANGEMENT

The types of gates and hoists commonly used for headworks of run-of-river projects are listed in Table 1.

Table 1: Types of Gates and Hoists

Location	Type of Gate	Type of Hoist	Remarks
Crest	Fixed wheel vertical/radial	Rope drum/Hydraulic hoist	Used in the spillway for discharge of flood. Vertical gates/radial gates chosen from consideration of factors like head, superstructure height and available width of pier ice conditions, etc. Limiting height of vertical lift: 8 m.
	Automatic gates	Float operated/counter weight operated hoist.	
	Stoplog gates which are fixed wheel-vertical gates or slide type	Gantry crane/Monorail crane automatically operated lifting beam.	These gates facilitate the maintenance of main crest gates.
River sluice	Service/ Emergency gate of fixed wheel type or slide type or radial gates	Screw hoist or rope drum hoist or hydraulic hoist.	Used to control the flow of water to river downstream side. For small heads, fixed wheel type gate operated with rope drum/screw hoists. Screwhoists limited to 15 tonnes. High head gates/ jet flow gates to be hoisted hydraulically.
Construction sluice	Fixed wheel vertical lift gates	Rope drum hoist/chain pulley blocks/winches/ movable cranes	Used for making construction sluice/ diversion tunnel dry, which has to be plugged after construction.

6. MATERIALS

The materials generally used for different parts of the gate are listed in Table 2. Any other material satisfying the requirements of the job may also be specified by the designer.

Table 2: Materials Generally Used for Gates

Component Parts	Recommended Materials
Skin plate, stiffeners, horizontal girders, Structural steel arms, bracings, tie members, anchorage girder, yoke girder, embedded girder, rest girder, load carrying anchors	Structural steel
Guide rollers and guide shoes	Cast iron, Structural steel, Forged steel
Trunnion, hub and bracket	Wrought steel, Cast iron, Cast steel, Structural steel
Wheels	Cast steel, cast iron, wrought steel, forged steel
Wheel pins or axles	Chrome nickel steel or corrosion resistant steel, mild steel with nickel or hard chromium plating
Wheel track	Stainless steel/corrosion resistant steel
Bearing/ Bushing	Anti-friction bearing/bronze, phosphor bronze, aluminium bronze, self lubricating bushing
Seal	Rubber
Seal seat	Stainless steel plate
Seal base, seal-seat base, seal clamp, sill beam	Structural steel
Guide	Structural steel/stainless steel
Springs	Spring steel / stainless steel
Anchor cables, rods	Structural steel

Grade of the material conforming to acceptable national or international codes shall be specified by the designer to suit the particular requirement. Steel for pin shall be electroplated with chromium as per the relevant codes.

7. DESIGN OF GATE COMPONENTS

General design of the vertical and radial gates involves design of the following components. Specific components are separately dealt with in relevant headings.

7.1 Skin Plate and Stiffeners

a. The skin plate and stiffeners shall be designed together in a composite manner.

b. The skin plate shall be designed for either of the following two conditions unless more precise methods are available:

   a. In bending across the stiffeners or horizontal girders as applicable, or.

   b. As panels in accordance with support conditions.

c. The stresses in skin plates for conditions in (7.1 b) shall be determined.

d. In either of the cases specified in (7.1 b) while designing the stiffeners and horizontal girders the skin plate can be considered coacting with them.

   The coacting width of the skin plate in non panel fabrication shall be taken by restricting to the least of the following values in mm:

   a. $40t + B$

      where, t is thickness of skin plate, B is width of the stiffener flange in contact with the skin plate.

   b. 0.11 span and

   c. Centre-to-centre of stiffeners and girders.

The width of the skin plate coacting with beam or stiffeners in panel fabrication as per (7.1 b.b) shall be worked out and stresses due to beam action calculated.

e. The stresses so computed shall be combined in accordance with the formula:

Eq. 1

$$\sigma_v = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \sigma_y + 3\tau_{xy}^2}$$

where, σ_V is combined stress, σ_X is sum of stresses along x-axis, σ_Y is sum of stresses along y-axis, and τ_{XY} is sum of shear stresses in x-y plane. The appropriate signs should be taken for σ_X σ_Y in the above formula.

f. The permissible value of mono-axial as well as combined stresses should not be greater than those specified (Refer IS 4622:2003).

g. Permissible value of stresses in welds shall be the same as permitted for the parent material. For site weld the efficiency should be considered 80 percent of shop weld.

h. To take care of corrosion, the actual thickness of skin plate to be provided shall be at least 1.5 mm more than the theoretical thickness computed based on the permissible stresses. The minimum thickness of the skin plate shall not be less than 8 mm, exclusive of corrosion allowance.

i. The stiffener may, if necessary, be of a built up section or of standard rolled section, that is, tees, angles, channels, etc.

7.2 Stiffeners and Girders for Vertical Gates

a. The horizontal and vertical stiffeners shall be designed as simply supported or continuous beam depending upon the framing adopted for gate. The spacing between main horizontal girders shall preferably be such that all the girders carry almost equal loads.

b. The end vertical girders shall be designed as continuous beam resting on wheel centre points with concentrated loads, coming from horizontal girders. Care shall be taken in carrying out the analysis of end girders to include torsional effect, if any.

c. The stresses in stiffeners and girders shall not exceed permissible values.

d. Whenever the gate is connected to the hoisting mechanism at points other than the end vertical girders, care shall be taken to avoid stress concentration particularly on the web of the top horizontal girder. The hoisting force should preferably be dispersed through suitable stiffeners to one or more horizontal girders below the top one. The extra stresses arising due to this arrangement may be combined with other stresses to ensure that the permissible limits are not exceeded.

7.2.1 Deflection of Vertical Gates

Maximum deflection of the gate under normal conditions of loading shall be limited to 1/800 of the span (centre-to-centre of the wheels).

However, in case of gates with upstream top seals a maximum deflection of the gate leaf at the top seal shall not be more than 80 percent of the initial interference of the seal.

7.3 Wheel and Wheel Tracks

The gate wheels shall be suitable to withstand the stresses developed due to hydrostatic loads, which they will carry. Wheels may preferably be without flanges but may be flanged, where considered necessary. For large spillway gates, the tread of the wheel or track may be slightly crowned to accommodate gate deflection under heavy load. The tread of the wheel may be flat when self-aligning bearings are used between the wheel and wheel pin.

The wheels shall be machined true to size and shall operate smoothly without vibration and without under going undue drift.

7.3.1 Design of Wheels with Point Contact

The capacity of the wheels shall be calculated and the maximum shear stress in N/mm^2 thus computed shall not exceed 2.41 x BHN where BHN is the Brinell Hardness Number of wheel tread or wheel path whichever is smaller, or 0.7 times ultimate tensile strength, whichever is less. In general, the required tread hardness shall penetrate to at least twice the depth at which the maximum shear stress occurs. The tread width/track width shall be such that with the gate deflected under the actual design loads, the distance from the edge of the wheel tread/track shall be at least 15 mm. The radius of crowning of the wheel or track shall not be more than 10 times the radius of the wheel.

Permissible values of contact stress as given as 1.4 times ultimate tensile strength shall be followed.

7.3.2 Design of Wheels with Line Contact

The contact stresses between the wheel and the track shall be calculated in accordance with the following formula:

Eq. 2

$$f_c = 0.418\sqrt{\frac{PE}{rl}}$$

where f_c is the contact stress in N/mm^2, P is the wheel load, in N, E is the modulus of elasticity, in N/mm^2, r is the radius of wheel in mm and l is the tread width in mm.

Permissible values of contact stress as given as 1.6 times ultimate tensile strength shall be followed.

7.3.3 Wheel Pins

The wheels shall be mounted on fixed pins. The pin shall be harder than bronze bushing. Grease fittings shall be provided to permit greasing of the bearings at easily accessible location and suitable grease holes shall be provided in the pin for this purpose.

a. The wheel pin shall either be supported at both ends, on one side of the web of the vertical girder and on the other side by the stiffener plate or of cantilever box of the end vertical girder. In the latter case the rigidity of cantilever box should be ensured.

b. The wheel pin shall be designed for bearing, bending and shear; the load shall be taken as the wheel load acting on the width of the bearing. The pin supports shall be suitably stiffened against bearings and tearing. The stresses shall not exceed permissible values.

c. The pins may have a suitable eccentricity to permit alignment of wheels, normally 5 mm.

7.3.4 Wheel Bearing

The wheel bearing may be bronze bushing, self-lubricating bushing or anti-friction roller bearings of any suitable design to suit the operational requirements and installation of gates. Where bronze bushing is used, the bearing stress shall not exceed the permissible values.

a. For antifriction roller bearings the outer diameter of the roller bearing shall not exceed 0.6 times the wheel diameter in case of point contact and 0.8 times the wheel diameter in case of line contact. The bearing shall be selected on the basis of factor of safety of 1.5 on the static capacity.

b. The following formula shall be used for computing the wheel frictional force for bush and roller bearing:

Eq. 3 $$F = \frac{P}{R}(f_a r + f_r)$$

where F is the total wheel friction in N, P is the total hydrostatic load in N, R is the wheel radius in mm, f_a is the coefficient of axle friction (sliding), f_r is the coefficient of rolling friction and r is the effective radius of bearing in mm.

i. The following values shall be used for coefficient of axle friction (sliding):

For	*Starting*	*Running*
Bronze bushing	0.20	0.15
Roller bearing	0.015	0.01

ii. For rolling between wheel and wheel track, the coefficient of friction shall be 1.0.

7.3.4.1 Fits and Tolerances

When bronze bushing is provided, the bushing shall be force-fit in the wheel and the wheel pin shall be running-fit in the bushing. When roller bearing is provided, the outside diameter of bearing shall be tight-fit in the wheel and the pin shall be tight-fit in the inside diameter of bearing.

7.4 Seal and accessories

a. Seals shall be fixed by means of seal clamps and galvanized iron or stainless steel bolts/stainless steel screws so as to ensure a positive water pressure between the seal and the gate, and to bear tightly on the seal seat to prevent leakage. For reducing the seal friction fluorocarbon clad seals may be used. Edges of seal clamp adjacent to seal bulb shall be rounded.

b. Solid bulb music note type seals are recommended for medium head gates. Hollow/ solid bulb music note type or flab or premoulded L-type rubber seals are preferred for low head gates.

c. For regulating gates, the designer at his/her discretion may make the seals effective throughout the range of the travel of gates either by fixing the seals to the embedded parts or by providing a liner plate above in continuation of the top seal seats for the entire width of the gate and range of regulation.

7.4.1 Initial Interference

The seal interference of double stem and the projection of bottom wedge seals shall vary from 2 mm to 5 mm depending upon the requirement and type of installation at the discretion of the designer. Suitable chamfer shall be provided at the bottom of the skin plate and clamp plate to accommodate the bottom wedge seal in compressed position.

7.4.2 Seal Friction

For the purpose of calculating the frictional forces to overcome, the following friction coefficients shall be used:

For	*Starting*	*Running*
Rubber seal on stainless steel	1.5	1.2
Fluorocarbon on stainless steel	0.20	0.15

7.5 Guide Roller and Guide Shoes

a. Gate guide roller/shoes shall be provided on the sides of the gates to limit the lateral motion of gate to not more than 6 mm in either direction. The rollers shall be flanged and travel on steel plates or rails securely attached to anchor bolts. In case of rollers it shall be provided with bronze bushing or self-lubricating bushing turning on fixed steel pins. Suitable arrangement for lubrication of these rollers shall also be provided. Where necessary, counter guide rollers shall be provided to limit the transverse movement of gates.

b. A minimum of two guide rollers or shoes should be provided on each side of the gate to resist the transverse and lateral movement of the gate and at the same time to prevent the gate from jamming. A clearance of 3 mm to 6 mm between the guide rollers and guide surface is normally recommended. The guide rollers or shoes, should be structurally adequate to withstand the load, they are likely to be subjected to, depending upon the type of installation, hoist and hydraulic condition. Guide rollers may also be provided with suitable springs, whenever required. Guide rollers may be preferred for gates to be handled by lifting beams.

c. Suitable spring assembly may be provided beneath the guide shoes or guide rollers assembly to restore the gate to normal position after any deflection.

d. The guide roller/shoes shall be designed to the maximum loads to which they may be subjected during operation. A minimum load of 5 percent of the total dead weight of the gate is recommended for the design of each guide roller.

7.6 Wheel track and track base

a. The wheel track shall provide a true and smooth machined surface for the wheels to roll and transmit the loads through the wheels to the track base.

b. The hardness of wheel track surface shall be kept minimum 50 points Brinell Hardness Number (BHN) higher than that of the wheel tread to reduce wear. For gates, which may not be put to frequent use, the difference between the BHN of wheel and wheel track may be reduced suitably at the discretion of the designer.

7.6.1 Thickness of Track Plate (with Line Contact)

The thickness of track plate shall be calculated from the following formula:

Eq. 4

$$b = 1.55\sqrt{\left(\frac{P}{l} x \frac{r}{E}\right)}$$

where l is the tread width in mm, b is the half contact width in mm, P is the wheel load in N, r is the radius of wheel in mm and E is the modulus of elasticity in N/mm^2.

Depth to the point of maximum shearing stress, in mm, Z_1 = 0.786b

The thickness of the wheel track shall not be less than 6 times the depth to the point of maximum shearing stress, Z_1.

7.6.2 Thickness of Track Plate (Wheel Track with Point Contact)

The thickness of track plate shall be calculated by the following formula:

Eq. 5

$$t = \frac{1.27P}{2cf_t}$$

where t is the track thickness in mm, P is the wheel load in N, $2c$ is the track width in mm and f_t is the allowable track bending stress, in N/mm^2 (0.4 YP of track material).

The minimum thickness of track plate shall be 10 mm.

The track base shall be embedded in concrete. It shall be designed as a beam on elastic foundation. The stresses in concrete under the track shall be found from the following formula:

Eq. 6 $$p = 0.281P\left(\frac{E_c}{E_s I w^2}\right)^{1/3}$$

where p is the bearing stress in concrete in N/mm^2, P is the total wheel load in N, E_c is the modulus of elasticity of concrete in N/mm^2, E_s is the modulus of elasticity of steel, in N/mm^2, I is the moment of inertia of track base in mm^4 and w is the width of track base in contact with concrete in mm.

The edge distance of the bearing flange of track base from the groove face shall be determined on the basis of the following criteria:

a. The wider flange, in case of double-flanged track base, shall be considered as bearing flange for the purpose of transferring load from the track base to the concrete.
b. The minimum edge distance 'e' of the bearing plate flange shall in no case be less than 150 mm.
c. The load shall be assumed to be distributed at 45^0 dispersion.
d. The width of loaded area at the interface of primary and secondary concrete shall fully lie in the primary concrete. Clear cover of the reinforcement is to be neglected.

The length of influence of the parabolic distribution under the track base may be found from the following formula:

Eq. 7 $$L = 1.5x\frac{P}{wp}$$

where L is the length of influence under track base in mm, P is the total wheel load in N, w is the width of track in contact with concrete in mm and p is the stress in concrete in N/mm^2.

If pressure distribution under adjacent wheels overlaps, superposition of the pressure shall be adopted and checked for the worst condition.

The track base shall be checked for bending and shear also. Bending stress shall be calculated from the following formula:

Eq. 8 $$\sigma_b = 0.5\frac{P}{Z}\left(\frac{E_s}{E_c}x\frac{I}{w}\right)^{1/3}$$

where σ_b is the bending stress in N/mm^2, P is theload on roller in N, Z is the of section of the track base about the neutral axis, in mm^3; E_c is the modulus of elasticity of concrete, in N/mm^2, E_s is the modulus of elasticity of steel in N/mm^2, I is the moment of inertia of the track base about the neutral axis in mm^4 and w is the width of track base in contact with concrete in mm.

The flange of the track base shall be checked for bending. The web of the track base shall be checked for compression. Permissible stress in compression for web shall be taken as 85 percent of yield point for normal condition and equal to yield point for MWL/occasional load condition. The stresses in track base shall be within the permissible values.

7.7 Guides

The guides shall be fixed inside the groove in piers. The guide shall be flat plate or a rail section anchored into concrete for gates fixed with guide rollers. The thickness of the plate shall not be less than as given below:

Type of Gate	*Thickness of Plate, mm*
Low head gate	20
Medium head gate	32

The guide shall be suitable for the type of guide rollers or shoes provided on the gate. The guide shall continue to the full range of travel of the gate.

7.8 Seal seat, Seal Seat Base, Seal-base and Sill Beam

a. The minimum width of seal seat shall be 80 mm excluding the required chamfer.
b. Minimum thickness of stainless steel plate shall be 6 mm for low head gates and 8 mm for medium and high head gates.
c. The seal-seat shall be either welded or screwed with corrosion resisting steel screws to the seal seat base. The number of screws should be such as to give sufficient rigidity to the assembly and water tightness.
d. The seal-seat shall be finished smooth to present a smooth surface to the gate seal.
e. The seal seat base shall be embedded in concrete,
f. The seal-seat base on which the seal-seat is fixed shall be made up of plate or any structural section. Angles or flats are used to fix the seal-seat base to the embedded anchor bolts. After complete installation the side seals wall plates and the bottom seal-seat shall preferably be flush with the surrounding concrete surfaces.
g. The edges of side and top seal seat shall be rounded/chamfered to prevent damage to rubber seal during gate operation.
h. The sill beam shall be provided with stainless steel flats welded or screwed on the top surface with stainless steel screws. The minimum thickness of stainless steel flat shall be 6 mm after machining. The surface of the sill beam shall be machined smooth and made flush with the surrounding concrete.

7.9 Ballast

Suitable ballast in the form of dead weight shall be added for making the gate self-closing, when necessary. The ballast shall be in the form of cast iron/pig iron billets, concrete or any other suitable material and shall not dislodge from its position during the gate operation.

7.10 Anchorages or Anchor Plates

In order to ensure proper alignment of the embedded parts anchorages shall invariably be provided in two stages that is partly in the first stage concrete, with suitable block out openings, to hold the balance embedded parts in the second stage concrete.

a. For proper bonding of first stage and second stage concrete, suitable dowel bars should be provided and interface surface should be kept thoroughly rough. The anchor bolts in second stage concrete shall be with double nuts and washers. For adjustment purposes enlarged holes in the second stage embedded parts plates/joist sections webs and flanges shall be provided. Preferably anchor plates may be embedded in first stage concrete and anchor bolts welded subsequently.
b. The anchor bolts should not be less than 16 mm diameter. Washers shall invariably be used with the anchor bolts.

7.11 Tolerances / Non-destructive Testing of Welds

The tolerances for embedded parts and components of gates shall be within the permissible limits. The distance between the wall plates shall be true within a tolerance of 3 mm. The anchors shall be set in the block outs within a tolerance of 3 mm.

All butt welds in gate components shall be 100 percent radiographically tested for their soundness. However extent of radiography for joints in the skin plate alone could be limited to 10 percent of total weld length suitably selected. However, other tests like ultrasonic dye penetrant or magnetic particle test could be conducted for full length of butt joints in the skin plate. All butt joints in hoist bridge shall also be tested radiographically 100 percent for their soundness.

8. VERTICAL LIFT GATE

Vertical lift gates are used for outlet works, and spillway crest gates. Each type of gate used has its advantages and disadvantages and is designed to accommodate special requirements for closure and retention of hydraulic head.

Some of the main advantages of using vertical lift gates are ease of fabrication, considerably shortened erection time, and in most cases, shorter monoliths or supporting piers for spillways compared with those of radial gates. The load from the gate to the supporting pier or monolith is in one direction, simplifying the design of supports. One main disadvantage when using vertical lift gates that are under constant cyclic loading is that the main load-resisting frame relies on a tension flange or, in the case of an arch, tension tie. In these cases fatigue plays a major role in their design. The use of fixed-wheel, tractor, Stoney, or slide gates versus radial gates for spillways and outlets depends on head, size of gate, river-flow operational criteria, and economics.

8.1 Types of End Supports

End supports for vertical lift gates may be classified according to the method used to transfer the loads to the gate guides. The gate guides receive the main reaction component from horizontal loads.

8.1.1 Fixed Wheel

With this type of end support, the wheels revolve on fixed axles, which are either cantilevered from the body of the gate or supported at each end by the web of a vertical girder(s) attached to the gate frame. The wheels may also be mounted by pairs in trucks that carry the wheel loads through center pins to end girders attached to the gate frame. When gate hoisting occurs with no static head, this type of end support will usually be most economical. The fabrication is generally less costly than using tractor type end support, described in Section 8.1.2 below. When the gate is used for outlet works, this type of end support will receive higher point loads. This will cause a much higher bearing stress to the wheel and guides, as well as shear, bearing, and bending forces to the center pins and end girder. This type of end support is normally used in situations where the gate is used to control flows while under low static head as with spillway gates, the wheels normally rest in a wheel recess to prevent them from transferring hydrostatic loads. With the wheels in the recess, horizontal loads are transferred through an end-bearing shoe to the pier-bearing surface. Hence the wheels carry no hydrostatic load. Hydrostatic load is then transferred from end bearing shoes on the gate to the gate guides.

8.1.2 Tractor

This type of end support has at each side of the gate one or more endless trains of small rollers mounted either directly or members attached to or on the vertical end girder. These are more commonly found on emergency closure gates or gates that control flow under high head. Because load transfer is achieved by uniformly distributed bearing through the small rollers, they are able to withstand large horizontal loads while being lowered under full hydrostatic head. Their main advantages over fixed wheels are a lower friction component while hoisting under load, lower bearing stresses transferred to the guides and gate framing, and shear and bending not transferred to the gate through the axle. When compared to slide gates, the main advantage is reduced friction, which reduces the hoisting effort required for controlling of flow. This reduced friction also reduces the wear and maintenance compared with those of slide gate seal surfaces.

8.1.3 Slide

Slide gates use metal-to-metal contact for end support. A machined surface that is mounted to the front face of the gate bears directly against a machined guide surface in the gate slot. The two bearing surfaces also serve as the gate seal. Materials for the gate seal surface may include aluminum, bronze, or stainless steel. These types of gates are normally used in situations where a head cover is used to seal off the guide slot from the gate operator for submerged flow installations. They can be used for high heads. The bearing surfaces of the guides and slide bearings shall be machined to tight tolerances to maintain a seal for the gate. This requires tighter construction tolerances for installation of the guides and slide bearing than with tractor gates and fixed-wheel gates, which use J seals along a seal plate.

8.1.4 Stoney

Similar to a tractor gate, a Stoney gate uses a small train of rollers; however, the fundamental difference is that the roller axles are held in position by two continuous vertical bars or angles on either side of the roller. The load is transferred from a bearing surface on the gate, through the rollers, to the guide-bearing surface on the monolith. The entire roller train is independent from the gate and the guide, which allows free movement of the roller train. In order to maintain the roller train in its proper vertical position, it is common to use a wire rope support. The rope is fixed to a point on the gate, and is fixed to a point on the pier or monolith. Lateral movement is prevented by vertical bars or axles along the guide surfaces. A unique feature of this type of load transfer system is that axle friction component attributed to rolling friction. The main advantages of this type of gate support system are the same as those of the tractor gates.

8.2 OUTLET GATES

Often, lift gates are used for emergency closure of water intake systems or outlet works. Their normal operation is in the open position. They are not used for throttling flows; however, they are used to stop flow under operating conditions. They normally rest on dogging devices during normal operation. In emergencies, they are lowered into the closure slot to stop the flow water. Outlet gates use fixed wheel, tractor, and slide support systems.

The time saving would occur for gates used for outlet works. Normal use for these types of gates is a tractor gate due to its low friction during operation. The size of gate and head requirements determine the feasibility of slide, fixed-wheel, or tractor gates. Slide gates require precise machined tolerances on the seal surfaces from the gate to the bearing guides. This requires careful quality control during field installation. Wear and damage to the slide and bearing surfaces due to use and cavitation can require higher maintenance to the slide gate. It may be more cost effective to replace wheels, rollers, or seals on a fixed-wheel or tractor gate than to fill and machine the gate and bearing surfaces of a slide gate.

8.2.1 Framing Systems

Horizontal girder framing is the most common type of framing system used for outlet gates. They may be framed with plate girders or rolled shapes. Horizontal plate girders are the main force-resisting members of the gate. They consist of built-up plate elements forming the stiffened webs and flanges of the girder. The spacing of the girders will depend on the head requirements, the height of the gate, and the clear span. For short gates, it is not advantageous to vary the spacing of the girders; however, for taller gates where the change in hydrostatic loading will be more significant from the bottom sill to the top, it is more economical to vary the spacing. Varying the spacing will require additional attention to design of the intercostals and skin plate to compensate for the varying hydrostatic pressure and span between girders. The girders frame into end posts that transfer end shear from the girders to bearing, either on the gate guides or through the types of end supports. Intercostals are framed plates or structural shapes that span the layers of horizontal girders used to create two-way plate bending action for the skin plate. Diaphragms are used to provide continuity of the gate by distributing horizontal loads more uniformly and supporting and distributing vertical loads.

The main difference in framing compared with that of spillway crest gates is that outlet gates require a sloping bottom or flat bottom with lip extension on the downstream side to reduce downpull forces while operating with water flow.

8.2.2 Load Types

The following load types are applicable to vertical lift gates used for outlet gates:

8.2.2.1 Hydrostatic

The hydrostatic load H_s shall be determined based on site-specific conditions that account for the differential between headwater and sill bearing at the invert. Headwater is determined from reservoir regulation studies for the dam.

8.2.2.2 Hydrodynamic

Hydrodynamic loads for outlet gates shall account for water hammer.

8.2.2.3 Gravity

Loads resulting from deadweight *D*, ice *C*, and mud *M* shall be based on site-specific conditions. Mud loads shall include silt loads where applicable. Ice loads are considered as gravity loads; lateral loads from ice are not considered in the load combinations.

8.2.2.4 Operating Equipment

Q is the maximum inertial load that can be exerted by the operating machinery. This shall consider the inertial effects of the deadweight, and in the case of double- or multiple-section gates, the inertial effects of the hydrodynamic load, H_d, ice C, and mud M, while using the gate for passing ice and debris; the effects of friction and binding of seals, slides and wheels.

8.2.2.5 Earthquake

Design earthquake load shall be determined based on an operational basis earthquake (OBE). The earthquake load E shall be based on inertial hydrodynamic effects of water moving with the structure. Sloshing liquid forces are small and may be ignored. The vertical distribution of the initial hydrodynamic pressures acting on the gate shall be determined from Westergaard's equation:

Eq. 9
$$p = \frac{7}{8}\rho_w a_e \sqrt{Hy}$$

where p is the lateral pressure (N/m^2) at a distance y below the pool surface in m, ρ_w is the density of water in kg/m^3, a_e is the maximum acceleration of the supporting lock wall due to the OBE in m/sec^2, H is the constant pool depth m and y is the distance below the pool surface in m.

The lock wall shall be assumed rigid in determination of a_e and the assumed direction of a_e shall be perpendicular to the gate. The inertial forces resulting from the mass due to structural weight D, ice C, and mud M are insignificant to the effect of p and need not be considered. For overhead gates, the effects of E shall be applied to the towers.

8.2.2.6 Downpull

Downpull forces R shall be determined based on flow conditions and the shape of the gate. These shall be determined by hydraulic studies or extrapolation of data from previous testing.

8.2.2.7 Wind loads

Wind loads W shall be based on site-specific conditions and shall be applied normal to the projected surface of the gate. For submersible gates, wind loads need not be applied.

8.2.3 Load Cases

Outlet gates shall be designed using working stress method.

The most unfavorable effect may occur when one or more of the loads in a particular load combination is equal to zero. For each load combination the gate should be considered supported on either its fixed supports or by the hoisting equipment. Q or R should be taken as zero when resting on its fixed supports.

Eq. 10 $$D+(C+M)+(Q \text{ or } R)$$

Eq. 11 $$D+H_s+H_d$$

Eq. 12 $$D+H_s+E$$

where, D is deadweight load of the gate, C is weight of ice, M is weight of mud or debris, Q is maximum inertial effects of machinery forces, R is downpull forces, H_s is hydrostatic load due to differential head, H_d is hydrodynamic load due to water hammer and E is lateral seismic forces from adjacent water

8.2.4 Serviceability Requirements

Vertical lift gates shall be designed for an expected life of 50 years. Limiting values of structural behavior to ensure serviceability shall be chosen to enable the structure to function as intended for its design life.

8.2.5 Fatigue and Fracture Control

For outlet gates, the total number of loading cycles is based on the projected frequency of usage over the life of the gate. Generally, outlet gates are operated infrequently; hence the fatigue is not a contributing factor to the design of the gate. Where projected usage of the gate is expected to place the members and connections into fatigue stress, then the fracture control requirements shall be followed.

8.2.6 Welds

All new gates use some form of welded fabrication. It is very important to select the proper weld material and the proper welding procedures.

8.3 Spillway Crest Gates

For spillway crest, radial gates are preferred over vertical gates due to lower maintenance. When multiple-section vertical lift gates are required, the latching mechanisms can become inoperable unless continued maintenance is performed requiring maintenance activities. However, vertical lift gates are preferred to radial gates when the elevation of the maximum controlled pool is so far above the sill that excessively long piers would be required for radial gates or flood discharge or drift conditions are such that any obstruction to the flow below the bottom of the spillway bridge is undesirable, requiring the gate to be removed.

Spillway crest gates use a horizontal framing system. This type may be framed with plate girders or rolled shapes. Most spillway crest gates have a fixed-wheel end support system. Tractor and slide gates are infrequent.

8.3.1 Framing Systems

Horizontal plate girders are the main force-resisting members of the gate. For further description Section 8.2.1 above may be referred.

8.3.2 Load Types

8.3.2.1 Hydrostatic

The hydrostatic load H_s shall be determined based on site-specific conditions that account for the differential between headwater and sill bearing at the spillway crest. Headwater is determined from reservoir regulation studies for the dam.

8.3.2.2 Hydrodynamic

The hydrodynamic loads H_d shall be determined based on site-specific conditions for vertical loads from water flowing over sections of spillway gates. The amount of head flowing over the sections of the gate is determined from hydraulic studies and operational criteria for the structure.

8.3.2.3 Gravity

Loads resulting from deadweight D, ice C, and Mud M shall be based on site-specific conditions. Mud loads shall include silt loads where applicable. Ice loads are considered as gravity loads; lateral loads from ice are not considered in the load combinations.

8.3.2.4 Operational Equipment

Q is the maximum inertial load that can be exerted by the operating machinery. This shall consider the inertial effects of the deadweight, and in the case of double- or multiple-section gates, the inertial effects of the hydrodynamic load H_d, ice C, and mud M, while using the gate for passing ice and debris.

8.3.2.5 Impact

Spillway crest gates are subject to debris or ice impact I of 75 kN/m along the gate at the upstream water elevation. Impact loads need only be applied to main load-carrying members. Skin plates and intercostals need not to be designed for impact loads.

8.3.2.6 Earthquake

Section 8.2.2.5 above may be referred.

8.3.2.7 Downpull

Downpull forces R shall be determined based on flow conditions and the shape of the gate. These shall be determined by hydraulic studies or extrapolation of data from previous testing.

8.3.2.8 Wind loads

Wind loads W shall be based on site-specific conditions and shall be applied normal to the projected surface of the gate.

8.3.3 Load Cases

Requirements for load combinations are similar to outlet gates described above in Section 8.2.3.

8.3.4 Serviceability Requirements

They are similar to outlet gates described above in Section 8.2.4.

8.3.5 Fatigue and Fracture Control

Generally, spillway gates are operated infrequently; hence the fatigue is not a contributing factor to the design of the gate. Where projected usage of the gate is expected to place the members and connections into fatigue stress, then the fracture control requirements shall be followed.

8.3.6 Welds

All new gates use some form of welded fabrication. It is very important to select the proper weld material and the proper welding procedures.

8.4 OPERATING EQUIPMENT

The operating equipment for vertical gates is referenced here for general description.

8.4.1 Types of Hoists

Vertical lift gates use hydraulic or wire rope hoist systems. Wire rope hoists are used for spillway crest and outlet gates. They are more suitable for gates that have deep submergence requirements, applications that do not allow portions of hydraulic cylinders above the deck (shallow settings), or when hoisting loads are too large and economics makes hydraulic cylinders impractical.

8.4.1.1 Wire Rope Hoists

Wire rope hoists consist of drums and a system of sheaves and blocks that are driven through a motor and arrangement of shafts, speed reducers, and spur or helical gears. Motors may be electric or hydraulic driven. It is common to provide two speeds to permit lowering at approximately twice the raising rate. The hoisting equipment is normally located next to the gate or slot with controls located in the control room, governor control cabinets depending on the gate and its intended use.

Bull Wheels

Bull wheels are used in overhead lift gates as a friction drive for hoisting the gate. The bull wheel, motor, and gearing system are located in a tower, high enough to raise the gate to its full and open position. The wire ropes wrap over the top of the bull wheel in grooves, with one side of the wire ropes connected to the gate and the other end to a counterweight. The motor and gear system provide the mechanical effort required to hoist the gate. This type of drum system is advantageous when the hoisting loads are large.

Counterweights

These are used mainly in overhead type gates to offset the dead load of the gate to minimize the hoisting effort. The weight of the vertical lift gate will determine the mass of the counterweight required. It should be designed to compensate for adjustment of its mass to calibrate it with the weight of the gate once the system is in place. It is normal to have the gate/counterweight slightly unbalanced to allow the gate to close without power. Another method for reducing the lifting effort is with a series of drums and sheaves, which are selected to give the mechanical advantage desired.

Motors and Gear Boxes

Motors and gear boxes are the primary drives for wire rope hoist systems.

8.4.1.2 Hydraulic Hoists

Hydraulic hoists normally consist of a single acting cylinder, pumps, reservoir, controls, and piping. Recent applications use telescoping cylinders to accommodate deep submergence gates. One or two cylinders may be used, dependent on the hoisting requirement and economics. The arrangement may include the cylinder to be supported above the gate with the gate and cylinder rod hanging from the piston or the cylinder recessed within the gate.

8.4.1.3 Roller Chain Hoists

Roller chain hoists consist of the lifting chain, drive and idler sprockets, drive machinery, and counterweight. The roller chains are located in recesses in the lock wall. Roller chains are flexible about an axis parallel to the lock centerline and rigid about an axis perpendicular to the lock centerline. Near the top of each recess the lifting chain is redirected by an idler sprocket to the drive sprocket, which is located in a recess below the top of the lock wall. From the drive sprocket the lifting chain continues to a second idler sprocket at the top of a counterweight chase. From the second idler sprocket the lifting chain extends vertically to the counterweight. The chain connection to the gate leaf is a three-dimensional gimbal, which allows rotation about the axes both parallel and perpendicular to the lock centerline. Rotation of the connection point is allowed to prevent the lifting chain from being bent about its rigid axis when the gate leaf rotates. The connection points on the gate should be located at the end portions, at the approximate center of gravity of the gate. The drive machinery, located in a watertight recess at the top of the lock wall, consists of electric motor, open gear sets, and reducers. An advantage of roller chains is the positive drive connection over the drive sprocket, which does not require the space of a cable drum. Disadvantages include relative high cost of chains, frequent maintenance for lubrication, corrosion, and critical alignment required between sprockets.

8.4.2 Dogging Devices

Dogging devices (dogs) are usually mounted on grillages in recesses in the piers opposite the end posts of the gate. They pivot to permit retraction for clearance of the gate and are operated with push rods. Two or more dogs at each end of the gate slot may be required. The number and location of the dogs are determined by the operating requirements for discharge regulation and gate storage. The gate sections require dogging seats fabricated with structural or cast steel, welded or bolted on the end posts. The treads of cantilevered wheels may be used as dogging seats. Another type of dogging device consists of a cantilevered mild steel H-beam that retracts inside the gate at each end between the top and second girder web. The beam is located at the center of gravity of the gate in the upstream/downstream direction and runs through the end post to a reaction point at an interior diaphragm. The dogging beam is extended and retracted by using a bar as a manual lever extending through a hole in the top web and into a row of holes in the top of the dogging beam. The cantilevered end of the beam rests on bearing pads recessed in the piers. Design should account for twice the calculated dead load to allow for impact.

8.4.3 Lifting Beams

Lifting beams are normally provided for outlet gates and maintenance bulkheads. Because these gates are normally stored in a submerged condition, the lifting beam provides a latching and unlatching mechanism to lift the gate from the slot.

9. RADIAL GATES

Radial gates are one of the simplest, most reliable and least expensive type of crest gate for passage of large floods. They require no slots in the piers and have good discharge characteristics. The conventional radial gate is not suited for the passage of floating material unless fully open, which may involve waste of water. This drawback may be overcome by altering the conventional gate by adding a flap to the top of the gate or by making the gate submergible so that water may be passed over the top of the gate.

Size of gate shall be specified as the clear width of opening and the vertical height above the sill of the gate up to the Full Supply Level or the top of the opening as applicable.

Normally, the radial gate has an upstream skin plate bent to an arc with convex surface of the arc on the upstream side (Figure 1 & Figure 2). The centre of the arc is at the centre of the trunnion pins, about which the gate rotates. The skin plate is supported by suitably spaced stiffeners either horizontal or vertical or both. If horizontal stiffeners are used, these are supported by suitably spaced vertical diaphragms, which are connected together by horizontal girders transferring the load to the two end vertical diaphragms. The end beams are supported by radial arms, emanating from the trunnion hubs located at the axis of the skin plate cylinder. If vertical stiffeners are used, these are supported by suitably spaced horizontal girders, which are supported by radial arms. The arms transmit the water load to the trunnion/yoke girder. Suitable seals are provided along the curved ends of the gate and along the bottom. The upstream face of the gate rubs against the top seal as the gate is raised or lowered (Figure 3). Guide rollers are also provided to limit the sway of the gate during raising or lowering.

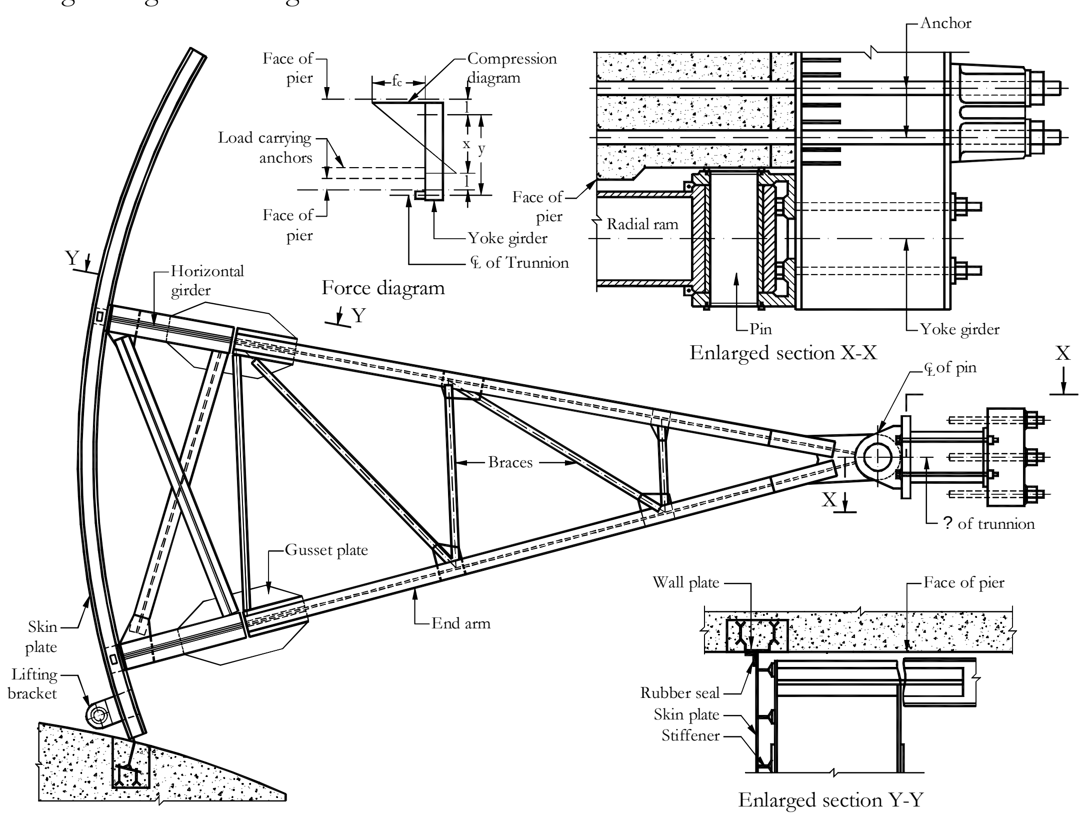

Figure 1: Radial gates with parallel Arm

The trunnion anchorage comprises essentially of a trunnion/yoke girder, held to the concrete of the spillway piers or the abutments by anchor rods or plate sections designed to resist the total water thrust on the gate. The trunnion or yoke girder is usually a built-up section to which the anchors are fixed.

i. The thrust may be distributed in the concrete either as bond stress along the length of the anchors (Figure 1) or as a bearing stress through the medium of an embedded anchor girder at the up stream end of the anchors. In the latter case the anchors are insulated from the surrounding concrete (Figure 2).

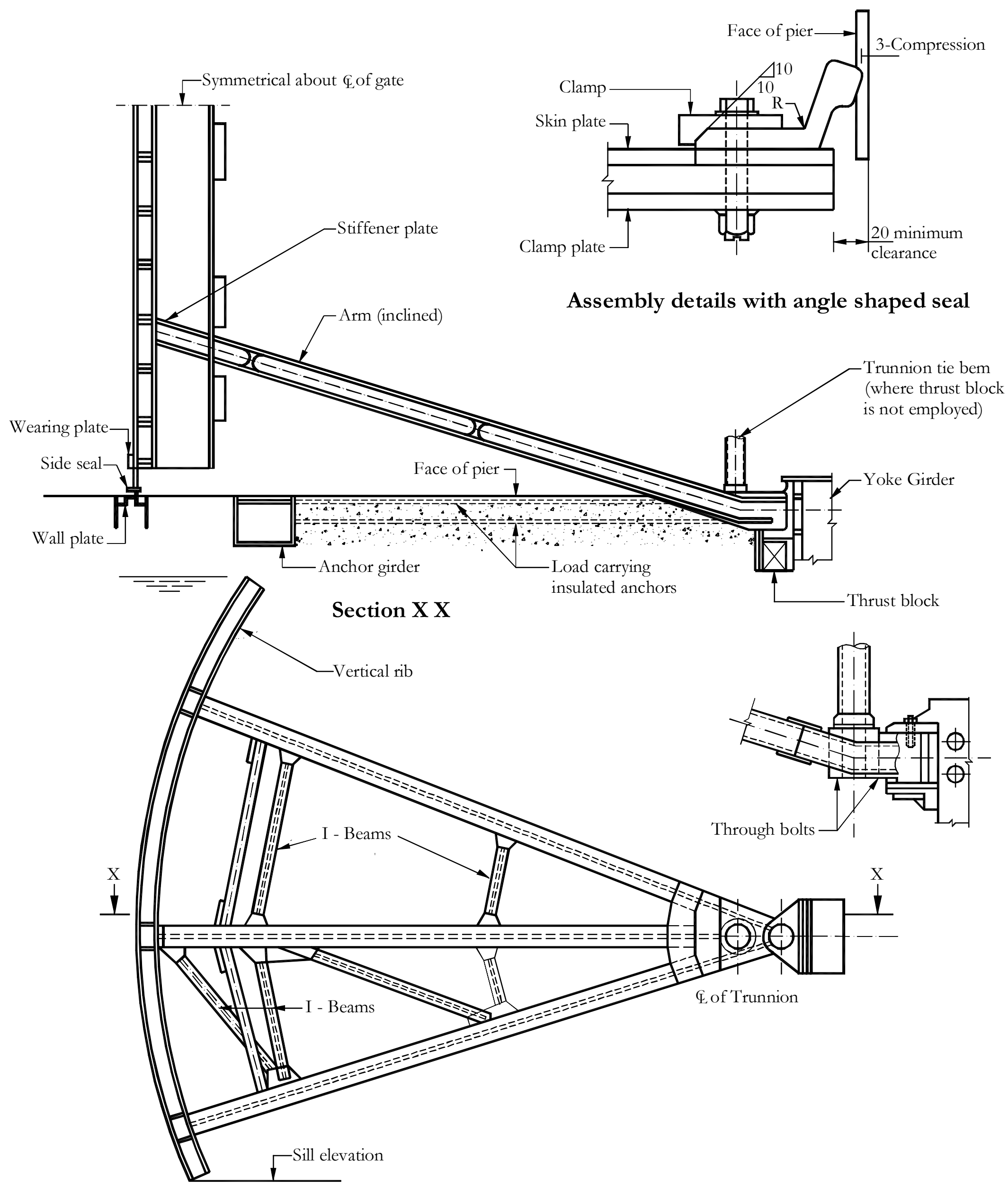

Figure 2: Radial gates with inclined arms

ii. Alternatively, anchorages of radial gates could also comprise pre-stressed anchorage arrangement. This system is especially advantageous in the case of large sized gates where very high loads are required to be transferred to the piers and the system of anchorages mentioned in (i) above is cumbersome and tedious. In this case pre-stressed anchorages post tensioned steel cables or rods are used which when subjected to water thrust will release pressure from concrete due to higher tensile stresses carried by anchorages. A typical arrangement is indicated in Figure 4.

The actual final stress developed in the cables/rods after allowing for all losses should not exceed 60 percent of the UTS or 80 percent of the YP of the material.

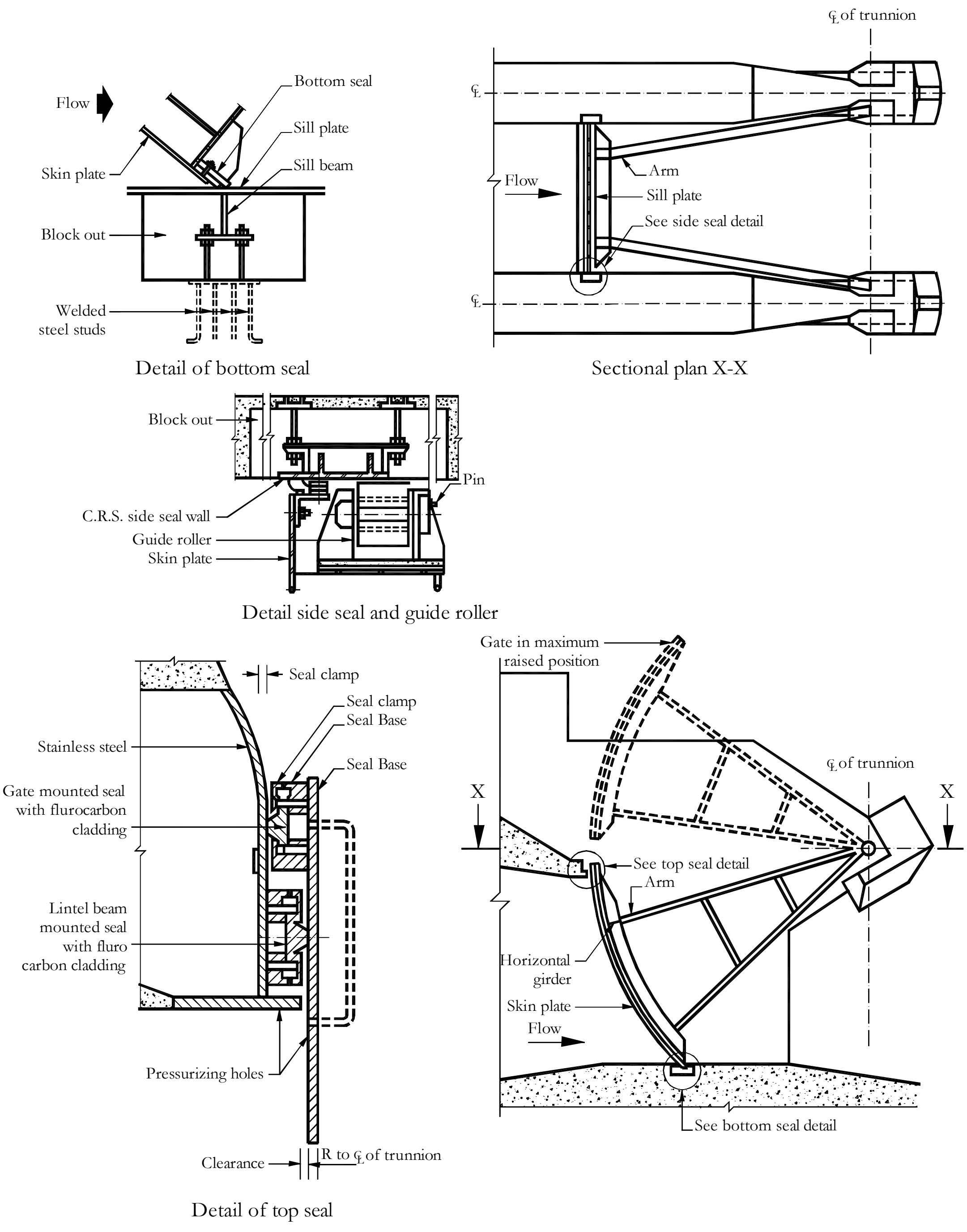

Figure 3: Sectional views of radial gates

iii. The grade of concrete as per concrete code around the anchorage system shall be:

- Conventional anchorage system M-25

- Pre-stressed anchorages system M-35

When the thrust is distributed in the concrete in bond, the anchorage girder itself is used to support the trunnion bracket. In the other case, another anchorage or support girder, in addition to the yoke girder supporting the load carrying anchors, is used to support the trunnion bracket.

If inclined radial arms are used instead of parallel arms, a side thrust block is provided to resist the side thrust. Alternatively a trunnion tie is also used for the same purpose (Figure 2) or the lateral thrust may be directly transferred to the concrete pier through bearing from plate embedded in the concrete.

Whenever occasional overtopping of gate is expected provision of hood, shield and flow breakers may be considered.

Hydraulic hoist operated flap gates may be provided at the top of large size radial gates for passing floating debris.

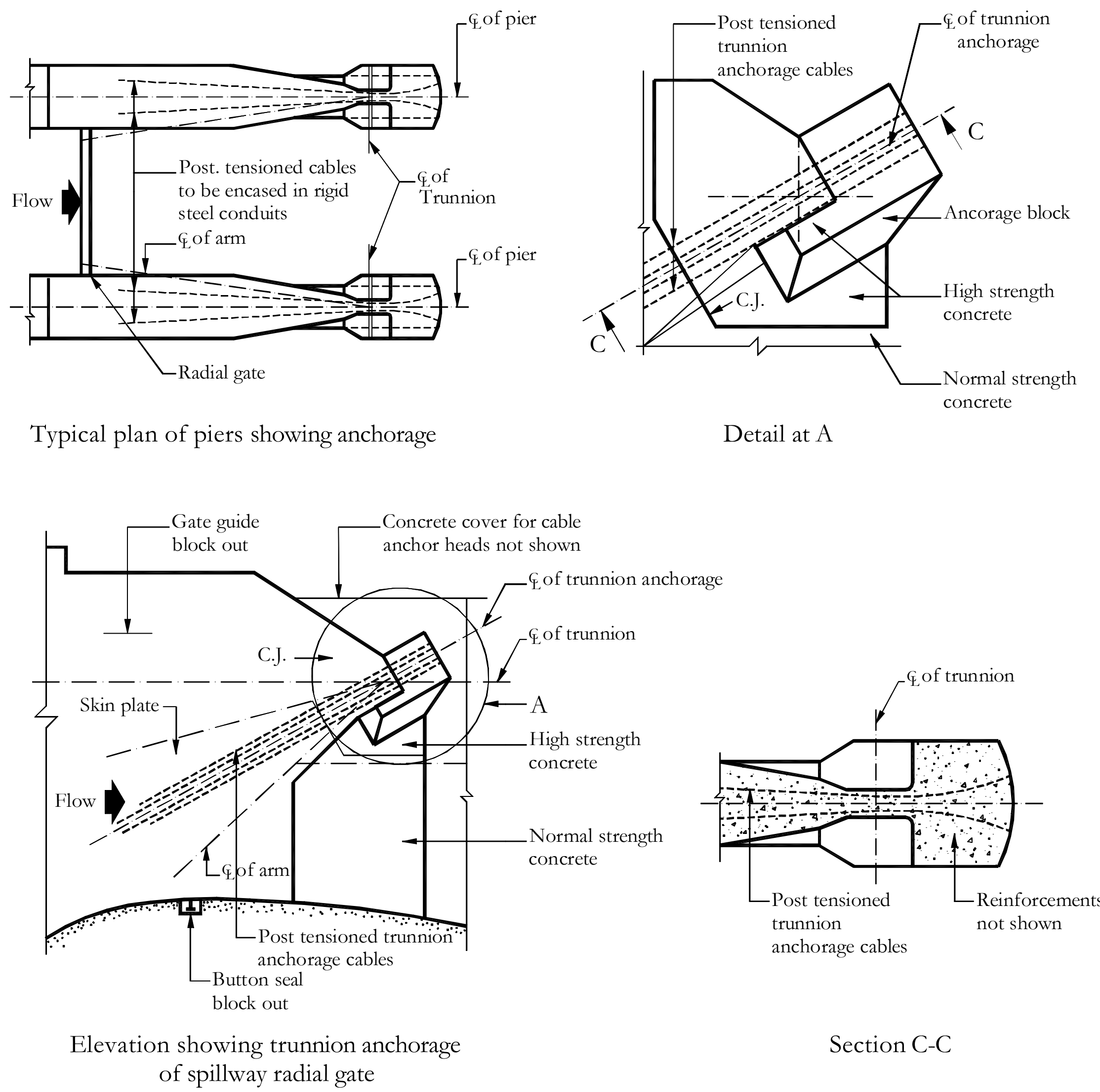

Figure 4: Elevation of trunnion anchorage of spillway radial gates

9.1 Design Considerations

The design of the radial gate involves the following parameters:

a. Location of the trunnions,
b. Radius of the gate,
c. Location of the sill, and
d. Location and type of hoists.

The design of the radial gate involves the following components. Those common are already explained and specific to the radial gates are briefly described in following pages.

a. Skin plate and stiffeners,
b. Horizontal girders,
c. Arms,
d. Trunnion hub,
e. Trunnion pin,
f. Trunnion bush or bearing,
g. Trunnion brackets,
h. Trunnion girder or yoker girder,
i. Load carrying anchors,
j. Anchorage girder,
k. Thrust block (if inclined arms are used),
l. Thrust tie(if inclined arms are used),
m. Seals,
n. Seal seat, seal base and sill beam,
o. Guide roller, and
p. Anchor bolts.

9.1.1 Location of the Trunnions

a. The trunnions of the gate shall be so located that under conditions of maximum discharge over the spillway, these should remain at least 1.5 m clear of the water profile and should in no case be allowed to submerge in the flowing water.
b. The trunnions shall be so located that the resultant hydraulic thrust through the gate in the closed position for reservoir full condition lies as close to the horizontal as possible. This will reduce the upward or downward force that will otherwise be imposed on the anchorage system.
c. The location of the trunnions shall be such as to allow the gate to be fully raised or lowered without interfering with the spillway or hoist bridge or any other part of the civil structure housing the gate.

9.1.2 Radius of the Gate

The radius of the gate, that is, the distance from the centres of the trunnion pins to the inside face of the skin plate shall, as far as possible, vary from H to 1.25 H consistent with the requirements of the trunnion location, where H is the vertical distance between the top of the gate and the horizontal line through the sill.

9.1.3 Location of the Sill

a. The sill of the gate shall preferably be located slightly downstream of the crest, to avoid cavitation of the downstream glacis.

b. The sill shall, as far as possible, be located so that a vertical plane tangent to the upstream face of the skin plate will intersect the spillway at or downstream from the crest. This requirement would place the sill downstream of the crest. Operating clearances from the bridge and the location of the hoist may require the sill to be shifted further downstream.

c. The requirement of Section 9.1.3(a) and (b) shall not apply and sill may be fixed as per the individual conditions.

d. The distance from the centre line of crest to the centre line of the sill shall be as small as possible in order to economize on the height of gate and pier size.

9.1.4 Location of Hoist

a. The radial gates are generally operated either by rope drum/chain hoists or by hydraulic hoist. For crest gates either of the two may be suitable.

b. In case of crest gates, the hoists may be installed on the roadway or on the piers or on an under-deck below the roadway.

c. The hoist shall be so located, that, as far as possible, the hoisting force is applied to the gate at the largest possible radius and the hoisting angle does not change much during the travel of the gate.

d. In the case of hydraulic hoist the connection to the gate is on the downstream of the skin plate while in the case of rope drum hoist the same is generally preferred on the upstream of the skin plate. In case of the rope drum hoist the hoisting connection can also be located on the downstream side of the gate depending on the site requirements. However, in such a case a minimum of two ropes shall be connected to each drum.

9.1.5 Horizontal Girders

a. The number of girders used shall depend on the total height of the gate but shall be kept minimum to simplify fabrication and erection and to facilitate maintenance. As a general thumb rule the number of horizontal girders and correspondingly number of arms may be adopted as follows:

   - For height of gate up to 8.5 m 2 no.
   - For height between 8.5 m and 12 m 3 no.
   - For heights above 12 m 4 or more

b. In the case of the vertical stiffeners designed as a continuous beam the girders may be so spaced that bending moment in the vertical stiffeners at the horizontal girders are about equal.

c. When more than three girders are used, it may become necessary to allow the bending moment in the vertical stiffener at the top most girder, of a value higher than at the other girders, so as to adequately stress the skin plate.

d. The girders shall be designed taking into consideration the fixity at arms support. Where inclined arms are used, the girders should also be designed for the compressive stress induced.

e. The girders shall also be checked for shear at the points where they are supported by the arms. The shear stress shall not exceed the permissible value.

9.1.5.1 Stiffeners and Bracings for Horizontal Girders

a. The horizontal girder should also be suitably braced to ensure rigidity.

b. The spacing and design of the bearing and intermediate stiffeners shall be governed by relevant portions of code of practice for general construction in steel.

9.1.6 Arms

a. As many pairs of arms as the number of horizontal girders, shall be used, unless vertical end girders are provided.
b. The arms may be straight or parallel. Inclined arms may conveniently be used to economise on the horizontal girders where other conditions permit.
c. The arms shall be designed as columns for the axial load and bending moment transmitted by the horizontal girders taking into consideration the type of fixity to the girder.
d. The total compressive stress shall be in accordance with relevant code of practice for general construction in steel for various values of l /r where l is the effective length, and r is the least radius of gyration. These stresses shall be further reduced by an appropriate factor depending upon the permissible stresses since the stresses are based on permissible stresses of 0.66 YP. For bending stresses, the stresses specified in relevant codes shall apply.
e. The arms if inclined may be fixed to the horizontal girders at about one-fifth of the width of the gate span from each end of the girder consistent with the design requirements.
f. The joints between the arms and the horizontal girders shall be designed against the side thrust due to the inclination of the arms, if inclined arms are used.
g. The arms shall be suitably braced by bracings in between the arms. The bracings connecting the arms that shall be so spaced, that the l /r ratio of the arms in both the longitudinal and transverse directions is nearly equal.
h. In case of gates likely to be overtopped, the end arms and other components should suitably be protected by means of side shields to prevent direct impact of water on arms. A hood may also be provided to protect the horizontal girders and other downstream parts.

9.1.7 Trunnion Hubs

a. The trunnion hubs shall rotate about the trunnion pins. The arms of the gate shall be rigidly connected to the hubs to ensure full transfer of loads.
b. The hubs shall be sufficiently long so as to allow the arms of the gate to be fixed to the respective limbs of the hubs, without having to cut and shape the flanges of the arms.
c. The thickness of the webs and flanges of each of the limbs of the hub shall be greater than that of arms to the extent possible so as to provide adequate space for the weld.
d. To ensure rigidity of the trunnion hub, sufficient number of ribs and stiffeners shall be provided in between its webs and flanges.
e. Minimum thickness of steel hub to be provided may be calculated from the following relationship:
    - For shaft up to 450 mm diameter t=0.3 d
    - For shafts above 450 mm diameter t=0.25 d subject to a minimum of 135 mm where, t is hub thickness, and d is diameter of the pin.

However, in the case of large size gates the hub may be designed as thick cylinder.

9.1.8 Trunnion Pins

a. The trunnion pin shall normally be supported at both ends on the trunnion bracket, which is fixed to the anchorage or support girder. Where convenient the trunnion pin may be cantilever from the anchorage box, embedded in the piers or abutments.

b. The trunnion pin may be solid or hollow and shall be designed against bending for the total load transferred through the trunnion hub. The load shall be taken to be uniformly distributed over the length of the pin bearing against the hub.

c. The trunnion pin shall also be checked against shear and bearing due to the same load.

d. The bending, bearing and shear stress in the trunnion pin shall not exceed 0.33 YP, 0.35 UTS and 0.25 YP respectively.

e. Provision shall be made for a grease hole on the outer surface of the trunnion so as to allow the trunnion bearing and connected grooves to be greased periodically.

f. The trunnion pin shall be medium fit in the bearing lugs of the support and shall be suitably locked against rotation.

g. The trunnion pin should be subjected to ultrasonic/radiographic tests to ensure soundness against manufacturing defects.

h. Trunnion pins should be hard chromium plated to a minimum thickness of 50 microns if the same is not made of corrosion resistant steel.

9.1.9 Trunnion Bush/Bearing

a. Depending upon the requirements of design and factors like accessibility after erection design, load, size, cost effectiveness, simplicity, dependability and other considerations any of the following may be used.
   - Slide type bronze bushing or self-lubricating bush bearings.
   - Antifriction roller bearings.
   - Other special type of bearings like spherical plain bearings.

b. The fits and tolerances to be adopted between the bushings/bearings, pin and trunnion hub shall be as follows.

Type of Bearing	Type of Fit	Machine Tolerance
Bronze Bushing		
a) with hub	Interference fit	H7r6
b) with pin	Clearance fit	H7f7
Antifriction Bearing		
a) hub	Interference fit	H7
b) Pin	Interference fit	r6
Spherical Plain Bearing		
a) hub	Interference fit	H7
b) pin	Interference fit	r6 or p6

c.

d. Minimum thickness of the bronze bushing to be provided can be determined by the following formula:

Minimum thickness of bushing in mm = 0.08 d + 3

where, d is the pin diameter in mm.

However thickness of bushing shall not be less than 12 mm. Other bearings may be selected as per manufacturers rating catalogues.

9.1.10 Trunnion Bracket

a. The bracket shall be rigidly fixed to the anchorage or support yoke girder by bolts or welding. It shall transfer the total load from the trunnion to the anchorage.

b. The bearing stress shall not exceed the permissible value.

c. The arms of the bracket shall also be designed to any bending, which may come on them due to the component of the load parallel to the base of the trunnion bracket.

d. Ribs and stiffeners shall be provided on the trunnion bracket, particularly on the sides of the bracket arms to ensure sufficient structural rigidity.

9.1.11 Anchorage System

a. The anchorage system shall be designed to withstand the total water load on the gate and transfer it to the piers and the abutments.

   Alternatively the trunnions may be located on an in-situ cast concrete beam/mass concrete in between the piers or concrete cantilever brackets transferring the loads directly in bearing.

b. The load may be transferred to the civil structure either in bond as a bond stress between the anchors and the concrete (Figure 1) or in bearing as a bearing stress between the concrete and the embedded girder at the upstream end of the anchors, which in this case are insulated from the concrete (Figure 2) or through a pre-stressed anchorage system using either steel rounds or steel cables.

c. Where the load is transferred by bond stress, rods are generally used as load carrying anchors. For insulated load carrying anchors, any convenient structural shape may be used although flats placed vertically or rods are generally preferred depending on the quantum of load. In the case of pre-stressed anchorages these can be either rods or cable.

d. For determining the force to be borne by the load-carrying anchors, the procedure as outlined in (i) and (ii), and (iii) and (iv) below may be adopted.

   i. The maximum horizontal and vertical force on the trunnion pin shall be determined. For this, the horizontal and vertical forces on the trunnion pin shall first be determined for the following two conditions:

      - Gate resting on sill and head on the gate varying from zero to maximum;
      - Water level constant at the maximum level for which the gate has to be designed and the gate position varying from fully closed to fully open. The worst combination of horizontal and vertical forces shall then be chosen; and
      - For combined anchorage the loading shall also be determined with one gate closed and adjacent gate fully opened.

   ii. If anchors used are inclined to the horizontal by an angle *m* the horizontal force so determined shall be multiplied by sec *m*.

   iii. For anchorages, where the anchors are not in a vertical plane through the trunnions but are in a vertical plane at a distance from the trunnions, the force F in the anchors shall be:

$$F = \frac{py}{x}$$

where, p is the force and x and y is the distance of the centre of gravity of the area in compression in concrete from the centre line of the load carrying anchors and the centre line of the trunnion respectively (Figure 1).

iv. For anchorages where the anchors are in a plane passing through the centre line of the trunnion and the thrust is transferred from the trunnions to these anchors through the yoke girder (Figure 2) the force in the anchors shall be P, where, P is the force determined in 9.1.11 (d) (ii) above.

v. The total stress in the anchors shall be the sum of the direct stress and the stress caused by the turning moment of the vertical force determined in 9.1.11 (d) (i) wherever applicable.

vi. The stresses in anchors made of structural steel shall not exceed the permissible values.

vii. If the load-carrying anchors are not welded to the trunnion girder but are fixed by nuts and locknuts, the ends of the anchors where considered economical may be forged to a larger diameter to provide at least the clear cross-sectional area of the anchors, at the root of the threaded portion.

viii. The length of embedment of anchors for bonded type anchorages shall be such that the bond stress shall not exceed the permissible values for the concrete used subject to a minimum of two-thirds of radius of gate leaf. Anchors may be hooked at the end, or alternatively provided with anchor plates. Dimensions of the hook shall conform to specification for bending and fixing of bars for concrete reinforcement.

- In case of bonded anchors to avoid cracking of face concrete, they should be insulated to a minimum of 500 mm length from the concrete face.
- The length excluding anchor girder of insulated anchors, which have an upstream embedded girder and where the load is transferred in bearing shall be such as to limit the shear stress in the 45° planes at the embedded girder to a safe permissible value subject to a minimum of 0.6 of radius of gate.

ix. All load carrying anchors whether bonded or insulated shall be suitably pre-tensioned on the trunnions to ensure proportionate load sharing by the anchor rods. In case of these anchors the pre-stresses shall be of a magnitude to introduce a stress of 5 percent of the permissible tensile stress in anchors.

9.1.12 Trunnion Girder or Yoke Girder

a. The trunnion girder may or may not be embedded in concrete. However, if embedded in concrete in the case of unloaded anchorage it shall also be wrapped in cork mastic or thermocole or such other material to provide space for displacement due to the loading. It shall support the trunnion bracket and be held in place by the load-carrying anchors (Figure 1 & Figure 2).

b. The girder shall be designed so as to be safe in bending, shear, and torsion if any caused by the forces.

c. The maximum shear stresses shall be calculated from the following considerations:

   i. That caused by horizontal and vertical forces respectively determined in Section 9.1.11(d) (i) and (ii), and

   ii. That caused by the torque at the centre line of the trunnion girder due to the vertical force at the trunnion for case stated under Section 9.1.11(d) (i),

   The total shear shall be the sum of values determined in Section 9.1.12 (c) (i) and (ii).

d. The maximum bending stress shall be calculated owing to the bending moment caused by the horizontal and vertical forces respectively.

e. The total shear stress calculated in Section 9.1.12 (c) shall be combined to the bending stress calculated in Section 9.1.12 (d) in accordance with code of practice for general construction in steel.

f. The total combined stress in the trunnion girder for either the web or the flange shall not exceed the permissible values.

g. The maximum vertical force calculated in Section 9.1.11(d) shall be distributed by the trunnion girder to the concrete below the girder.

h. The total compressive stress in the concrete shall be determined by combining the direct compressive stress with the compressive or tensile stress caused due to the eccentricity of the vertical force.

   The concrete immediately in contact with the trunnion girder which takes the thrust in bearing from it should be of non-shrinkage quality for a minimum thickness of 300 mm.

i. The maximum compressive stress in bearing at any point in the concrete in contact with anchor plate/girder shall not exceed 0.25 f_{ck} where f_{ck} is compressive strength, at 28 days, of the concrete used.

j. Where the horizontal force from the trunnion pin is directly transferred to a yoke or trunnion girder immediately behind the trunnion pin (Fig. 2) the yoke or anchorage girder shall be designed against bending and shear caused by this force.

k. The girder shall be treated as a simply supported beam loaded at the centre and supported at the junction of the girder and the load carrying anchors.

l. The bending and shear stress in the girder shall not exceed the permissible values.

m. To allow for the elongation of the insulated load-carrying anchors and trunnion tie if used, the trunnion bracket shall be so fixed as to be able to slide on the rest chair.

   Bronze or stainless steel pads shall be used for this purpose both on the top of the rest chair and at the bottom of the trunnion bracket:

n. The bearing stress on the bronze or stainless steel pad shall not exceed 0.3 YP.

o. Invariably all welded girders should be stress relieved unless the maximum thickness of plate used is less than 36 mm.

9.1.13 Thrust Block and Trunnion Tie

a. The thrust block or trunnion tie is required only if inclined arms are used with the gate for resisting the horizontal forces. Alternatively, this lateral thrust can be directly transferred to the concrete pier through bearing from a plate embedded in concrete.

b. The thrust block shall be used when the horizontal force from the trunnion is directly transferred to a yoke girder immediately behind the trunnion Section 9.1.12 (j).

c. The thrust block is fixed to the trunnion/yoke girder and is designed to withstand the bending and shear force caused by the side thrust on the trunnion due to the inclined arms. A thrust washer should be used between the trunnion hub and trunnion bracket to transfer the thrust.

d. The effect of the thrust block shall also be considered while computing the total compressive stress as given in Section 9.1.12 (h).

e. To allow for the elongation of the insulated load-carrying anchors, bronze to bronze sliding surfaces or bronze to stainless steel sliding surfaces shall be provided on the face of the thrust block and the mating face of the trunnion bracket.

f. The bearing stress on the bronze or stainless steel pads shall not exceed 0.3 YP.

g. Alternatively the trunnion tie can be used to withstand the side thrust caused by use of inclined arms.

h. The tensile stress in the tie beam shall not exceed the permissible values.

i. The trunnion tie shall span from one trunnion hub to the other and shall be fixed securely to the trunnion hub either by welding or by long bolts passing through holes in the trunnion hub.

j. For the trunnion tie beam standard rolled/fabricated section or a steel pipe with flanges at the ends for bolting may be used.

k. The deflection or the trunnion tie beam due to self-weight shall be in the permissible range.

10. FLAP GATES

This type of gate is a leaf hinged at bearings along its lower edge. The leaf may be flat or curved to give better discharge characteristics when rotated to its open position. The position of the leaf may be controlled by hoisting attachments that pull or push at one or both ends or by hydraulic or screw-stem hoists that push at selected locations under the gate. This type of gate can be built to great lengths and is well suited for passing floating material and for close regulation. Counterweights and/or floats may be incorporated in the hoisting mechanism of relatively small flap gates to provide automatic operation with little or no other source of power.

The flap gates normally operate at partially open conditions and shall be designed for the hydrodynamic effects of the overflowing sheet of water. If not properly designed and vented, destructive vibrating forces may occur. It is recommended that hydraulic-model studies simulating all expected opening conditions of the prototype gate be carried out before proceeding with the fabrication of flap gates of any importance.

PART 2J – HYDRAULIC MODELING

TABLE OF CONTENTS

Part

2J

Hydraulic Modeling

1. PURPOSE OF GUIDELINES

Part 2J of the *Design Guidelines for Headworks of Hydropower Projects* provides guidance for the hydraulic modeling of headworks of run-of-river hydropower projects.

2. SCOPE OF GUIDELINES

The guidelines cover the need of physical modeling, design, construction and for different components of headworks of run-of-river projects. They also contain a brief discussion on numerical modeling of such structures.

3. TERMINOLOGY

Terms and abbreviations used in these guidelines are defined below:

Froude number	A dimensionless number expressing the ratio between the influences of inertia and gravity in a fluid.
Hydraulic model	A physical scale model of a river or hydraulic structures used for engineering studies.
Model	A representation of a physical process or thing that can be used to predict the process's or thing's behavior or state.
Numerical model	Representation of a mathematical model as a sequence of instructions (program) for a computer.
Physical model	Model using the physical properties and behavior of modeling materials to represent the prototype.
Prototype	The full-sized structure, system, process or phenomenon being modeled.
Reynolds number	Dimensionless ratio of inertial force to viscous force.

Scale (or scale ratio)	Ratio of a parameter in a model to the corresponding parameter in the prototype.
Scale effect	Consequence of non similarity between model and prototype resulting from the fact that not all pertinent dimensionless numbers are the same in model and prototype.
Scaling laws	Conditions that must be satisfied to achieve desired similarity between model and prototype.
Similarity (or similitude)	Correspondence between the behavior of a model and its prototype.
Simulation	Reproduction of the prototype behavior using a model.

4. OBJECTIVES OF MODELING[1]

Physical model of the headworks of run-of-river hydropower projects shall be performed for the following purposes to:

a. Develop a conceptual layout of overall headworks structures in respect of abstraction of flow, exclusion of sediment load, protection against erosion and floods including environmental considerations.
b. Verify and optimise the initial design of hydraulic structures.
c. Display possible problems and demerits of the proposed hydraulic design so that other alternative solutions can be explored and optimised with simple modification in the model with low cost.
d. Provide insight and visual performance to facilitate realistic decisions on the proposed design. It is a tool for demonstration of proposed design to the decision makers. Sometimes it saves from the project investment failure. It is also useful to demonstrate all the stakeholders and address the public concern and environmental issues to some extent.
e. Develop headworks operation guidelines
f. Carry out research and development.

5. SCOPE OF MODELING

Followings are few issues in the headworks design of run-of-river hydropower project that can be studied from the physical model study:

a. Overall arrangement of diversion structures (dam/ weir), gates, intakes and settling basin,
b. Reservoir level and down stream water level for different flow conditions
c. Capacity of spillways, bed load exclusion, sediment excluders or bottom outlets
d. Design of stilling basin and energy dissipaters
e. Intake and spillway gate operation rules for flow management
f. Dynamic pressure at gates and other structures
g. Cavitation possibilities in spillway
h. Passage of floating material, ice or drifting debris
i. Fish ladder location and operation
j. Sediment accumulation and its removal process at peaking pondage
k. Performance of settling basins
l. Headloss in the headworks
m. Fulfill production regularity requirement

[1] Unless otherwise stated, the term "modeling" refers to "physical modeling"

6. NEED FOR MODELING

The complex nature of Himalayan Rivers in terms of hydraulics and sediment transport capacity pose a great challenge to the hydropower designers. The unavailability of consistent and ample hydrological records and lack of extensive research on the river behaviour including sediment yields resulted early failures and inefficient functioning of many headworks of existing hydropower projects. Therefore, a design of headworks in a hydropower project is different and river specific in Nepalese context. Based on past experience and knowledge, the theoretical analysis of River hydraulics in Nepal does not precisely represent their hydraulic behaviour. Hence, the hydraulic physical models are the only tools that can be used for the efficient design of headworks on such rivers. The physical models are used to predict and visualize the performance and effects of proposed design in a reduced scale laboratory experiments.

Most headworks structures of a hydropower project are surrounded by a fairly complex three dimensional flow field. The flow of water is generally mixed with sediments, in which the bed load and suspended particles move with the flow. A good hydraulic design of headworks is often essential for its performance. A better performing hydraulic design can only be opted if the performance of analytical design is visualized in a physical model study.

The necessity of physical modeling depends upon the complexity of headworks arrangement and designer/proponent's requirements. The location of the headworks, steep river gradient, complexity of the flow condition, high sediment load are the major causes that the flow regimes may not necessarily take place at the headworks as assessed on the theoretical design. Therefore, the requisite of physical modeling of a hydropower project depends upon the specific project and decision of the project designer.

7. MODELING PHILOSOPHY

A physical model is a simplified scaled representation of a hydraulic flow situation. Physical hydraulic models are commonly used during design stages to optimise a structure and to ensure a safe operation of the structure. It is based on the application of a set of relationships which are known as the laws of hydraulic similitude. The laws of similitude for flowing liquid are based on the relationships derived from Newton's second law of motion.

7.1 Law of Similarity

Basic principle of scaled model is governed by the theory of similarity between model and prototype. The theory of similarity shows:

a. how the model experiment should be theoretically founded and methodologically prepared;
b. what requirements the model must fulfil to depict reality on a reduced scale as reliable as possible;
c. which parameters should be measured during the experiment;
d. how the research results must be processed;
e. to what phenomenon the obtained results may be applied and what is the extent of their validity.

Dimensional analysis is the most powerful tool in experimental fluid mechanics, which allows in a simple and direct manner – implicitly – the formulation of criteria for dynamic similarity. Since all relationships derived by dimensional analysis are independent of the absolute scale, they must be applicable for both models and prototype structures. The theory of similarity, leading to dimensionless numbers and scaling laws, can be elaborated in three ways. The first determines the criteria of similarity from a system of basic differential equations in which the investigated physical phenomena are expressed mathematically. The second path leads to the condition of similarity through dimensional analysis carried out after a careful appraisal of the physical basis of each phenomenon and of the parameters which influence it. The combined use of physical and dimensional analysis is often the best route to a successful formulation of the similarity criteria. The third route could be denoted as the method of synthesis.

There are four types of similarity, which are employed for the scale modeling process. These are geometric, kinematic, dynamic and mechanical similarities.

If a model of any scheme is built corresponding exact shape of the prototype by reducing all dimensions and having the same corresponding angles, then this is referred to as a geometric similarity of the model and prototype. This is independent of motion of any kind and involves only the similarity in form. Similarity of length can be expressed as follows:

Eq. 1 $$L_r = \frac{L_m}{L_p} \quad \text{(Length)}$$

Where L_m and L_p are corresponding linear dimensions in model and prototype and L_r is the length ratio which is simply called the model scale. Similar ratios for area and volume can be expressed in terms of the model scale as following:

Eq. 2 $$A_r = \frac{L_m^2}{L_p^2} = L_r^2 \quad \text{(Area)}$$

Eq. 3 $$V_r = \frac{L_m^3}{L_p^3} = L_r^3 \quad \text{(Volume)}$$

Kinematic similarity implies that the ratios of model characteristics to prototype characteristic velocities are the same:

Eq. 4 $$v_r = \frac{v_m}{v_p} = \frac{(v_1)_m}{(v_1)_p} = \frac{(v_2)_m}{(v_2)_p} = idem \quad \text{(constant) (Velocity)}$$

Dynamic similarity implies that the ratios of model forces to prototype forces are equal:

Eq. 5 $$F_r = \frac{F_{1m}}{F_{1_p}} = \frac{F_{2m}}{F_{2_p}} = idem \quad \text{(constant)} \quad \text{(Force)}$$

Work and power are other parameters involved in dynamic similitude. Geometric similarity is not enough to ensure that the flow patterns are similar in both model and prototype (i.e. kinematic similarity). The combined geometric and kinematic similarities give the model-to-prototype ratios of time, acceleration, discharge and angular velocity. Mechanical similarity is that which includes all-embracing term including geometric, kinematic and dynamic similarity, i.e. L_r, v_r and F_r are all constant, the same in all directions.

7.2 Froude Law

All physical model studies with reduced model scale have to simulate the important parameters by neglecting some less important parameters. In practice, most of the physical hydraulic models are scaled with either Froude or with Reynolds numbers for hydrokinematic conditions, which means the selected dimensionless number is same in the model and in the prototype.

The Froude law represents the condition of dynamic similarity for flow in model and prototype, exclusively governed by gravity. Other forces such as capillary force, frictional resistance of a viscous liquid, the force of volumetric elasticity and cavitation phenomena, either do not affect the flow or their effect may be neglected. However, in the flow of real (viscous) fluid, internal frictional force always acts simultaneously with gravity. If the geometrical dimensions and boundary conditions of the model and prototype are similar to each other, similarity between forces due to gravity as well as resistance due to friction (in large extent) can be ensured.

Therefore geometric similarity is taken as the basis of mechanical similarity while investigating the similarity under exclusive action of gravity. Froude similarity is valid not only for the exclusive gravity but also for the other volumetric forces, e.g. centrifugal forces. On the other hand, modeling of motion according to Froude similarity is considerably simple, since the velocities of the flow on the model are smaller than those on prototype. Therefore, in most instances, Froude similarity is considered the best for the investigation of open channel flow on the physical models.

The headworks of hydropower project mainly include the hydraulic structures having free flow surfaces. In free surface flows, the presence of the free-surface governs the gravity effects as important influence. The Froude number which is expressed as $\left(Fr = \frac{v}{\sqrt{g.L}} \right)$, where v is velocity, g is acceleration due to gravity and L is characteristics length/depth. Few examples of secondary scale ratios that can be derived from the dimensional analysis and constancy of the Froude number are:

Eq. 6 $$v_r = \sqrt{L_r} \quad \text{(Velocity)}$$

Eq. 7 $$Q_r = v_r \cdot A_r = \sqrt{L_r} \cdot L_r^2 = L_r^{5/2} \quad \text{(Discharge)}$$

Important scale ratio for different parameters in the Froude similarity factors are shown in Table 1.

Table 1: Similarity scale factors

Parameter	Unit	Dimension	Scale Ratio with	
			Froude Law	**Reynolds Law**
Geometric properties				
Length	m	L	L_r	L_r
Area	m^2	L^2	L_r^2	L_r^2
Volume	m^3	L^3	L_r^3	L_r^3
Kinematic properties				
Time	s	T	$L_r^{1/2}$	$L_r^2\ \rho_r/\mu_r$
Velocity	m/s	LT^{-1}	$L_r^{1/2}$	$1/L_r\,(\mu_r/\rho_r)$
Acceleration	m/s^2	LT^{-2}	1	$1/L_r^3\ (\mu_r/\rho r)^2$
Specific discharge	m^2/s	L^2T^{-1}	$L_r^{3/2}$	μ_r/ρ_r
Discharge	m^3/s	L^3T^{-1}	$L_r^{5/2}$	$L_r\,(\mu_r/\rho_r)$
Dynamic properties				
Mass	kg	M	$L_r^3\rho_r$	$L_r^3\rho_r$
Force	N	MLT^{-2}	$L_r^3\gamma_r$	μ_r^2/γ_r
Density	kg/m^3	ML^{-3}	ρ_r	ρ_r
Specific weight	N/m^3	$ML^{-2}T^{-2}$	γ_r	$1/L_r^3 * \mu_r^2/(\rho r)$
Pressure intensity	N/m^2	$ML^{-1}T^{-2}$	$L_r\,\gamma_r$	$\mu_r^2/\rho_r\,(1/L_r^2)$
Dynamic Viscosity	N/m^2-s	$ML^{-1}T$	$L_r^{3/2}\rho_r^{1/2}\,\gamma_r^{\,1/2}$	μr
Surface tension	N/m	ML^2T^{-3}	$L_r^2\gamma_r$	$\mu_r^2/\rho_r\,(1/L_r)$

7.3 Reynolds Law

The Reynolds law represents the fully-enclosed flow conditions in which viscous forces are predominant. In this case, the basis for similitude is obtained by equating the ratio of viscous forces to that of inertial forces and neglecting the other forces. Fully-enclosed flow situations include pipe flows, valves and turbo-machines. In these situations, viscosity effects on the solid boundaries are important. The Reynolds number is expressed as $\left(Re = \frac{v.L}{\upsilon} \right)$, where v is velocity, g is acceleration due to gravity and υ is the kinematic viscosity. Thus the physical modeling of these structures is usually performed with a Reynolds similitude, i.e. the Reynolds number is kept identical in both the model and prototype:

$$Re_p = Re_m$$

In laboratory experiments making use of the same fluid in both the model and prototype, then the Reynolds model law reduces to the requirement that the velocity scale must be chosen inversely proportional to the length scale:

Eq. 8 $$v_r = \frac{1}{L_r}$$

This implies that in a small scale model the resulting velocities must be larger than in the prototype. The relationships for different parameters derived from Reynolds similarity scale factor are also presented in Table 1.

Although there exist the Weber number (proportional to ratio between inertial and surface tension) and the Euler Number (proportional to ratio between inertial and pressure force) as hydraulic model laws, they do not have significant importance in the physical modeling of headworks of hydropower plants. Therefore the present report does not explain further about these laws.

If the acceleration due to gravity g is one of the characteristic parameters, then we have to choose $g_r = 1$, as this is same for the prototype and model. Similarly for the same liquid in model and prototype $\gamma_r = \rho_r = \mu_r = \nu_r = 1$ for $\gamma_p = \gamma_m = \gamma$, where γ, ρ, μ and ν are specific weight, density, dynamic viscosity and kinematic viscosity of liquid, respectively.

8. SELECTION OF PHYSICAL MODEL SCALES

Selection of the physical model scales is one of the delicate tasks during physical modeling process. Dimensional analysis is a useful means for organizing and understanding the problem and for setting up a scaling framework. The modeler must check for scale effects and build a model that simulates best those aspects of the prototype, which are of prime interest. This means that several models with different model scales are necessary to study different aspects of one prototype process within the headworks.

Before building a physical model, model specialists must have the appropriate topographic and hydrological field information. The type of model must then be selected, and a question arises: Which is the dominant effect: e.g. viscosity, gravity or surface tension? The answer of the question sets the criteria for the adoption of similarity law. Other elements to be considered are the limits of the modeling facilities (maximum discharge, maximum head, floor area, and ceiling height), construction considerations, instrumentation limitations, and scale effects. A model's geometric scales may be selected so as to fit laboratory space constraints; and required equipment available in the market.

8.1 Scale Limitations

Assuming the model is properly designed, constructed, and operated, the scaling criteria provide mathematical relationships that can be used to determine corresponding model and prototype values of these parameters. Particular discrepancies between the model and prototype behavior are caused by the scale effects.

Some of the most common causes of scale effects are observed on cavitation, friction, and surface tension phenomenon. Classic examples of cavitation and friction-related scale effects arise in physical model of small orifice holes. Surface tension can affect Froude-scaled models if the depth becomes lower than about 25 mm. The scale effects lead to some guidelines for scale limits on various types of models. The Bureau of Reclamation (USBR) has recommended certain length scale ranges for different hydraulic structures built in hydropower, river and estuaries.

The model should be designed as large as practicable, considering cost and benefits. Few examples of the range of such model scales for the design of physical models of headworks of hydropower project are presented below:

a. Model scale (L_r): 30-100 for models of spillways of diversion weirs/dams.
b. Model scale (L_r): 5-30 for settling basin, stilling basin and outlet works having gates and valves.
c. Model scale (Lr): 3-20 for, side channel spillway chutes, drops and canal structures.

In addition to above mentioned suggested scale ratios, certain minimum dimensions must also be maintained for successful studies. The modeler should keep in mind the size of structures in prototype while selecting the scale ratios. Models of gates and conduits should have passages of at least 100 mm across, so that, for the head and discharge typically available in the laboratory, turbulent flow is produced in the pipeline. In models of canal structures, the bottom width of the channel should also be at least 100 mm. To minimize the relative influence of viscosity and surface tension, spillway models should be scaled to provide flow depths over the crest of at least 75 mm for the normal operating range.

8.2 Experimental Limitations

The available experimental facilities are the main limiting factors for the design and sizing of models. The size of models shall be such that the maximum physical conditions are provided. In addition to this, attempt shall be made to anticipate expansion of space and flow rates that might take place during the course of the study.

8.3 Instrumentation Limitations

Since the objective of every model investigation is a carefully planned series of measurements, the required instruments comprise an essential feature of the laboratory equipment. The size and sensitivity of the instruments are the major limitations influencing the physical model study. The position of instruments should be at the accessible locations for observations, with enough clearance between instruments and flow boundaries to give accurate measurements. The selected scale must be such that the magnitude of measured quantities is well within the range of available instruments and the sensitivity of the instrumentation is sufficient to obtain the results for different operating conditions.

8.4 Construction Considerations

Very small discrepancies in the model can result in large differences, when transferring them to the prototype structure. Therefore the scale factor and construction techniques must be chosen so that the required accuracy and precision can be maintained in the construction of model features. Smaller the model scale the higher will be the discrepancies, thus it is necessary to obtain the reasonably representative model size. Similarly, the location for the installation of instrumentation shall be carried out accurately to obtain proper results. Parts of the model which are likely to undergo changes to obtain results shall be constructed such that they can be dismantled without disturbing the whole model.High quality workmanship is also equally important during the model construction.

9. PHYSICAL MODEL INSTRUMENTATION

The foremost objective of instrumentation is to provide the necessary information to compare design alternatives, predict prototype performance, or develop generalized results applicable to a wide variety of situations. Many physical model studies involve the extensive use of simple instruments, such as point gauges, pitot tubes, and manometers for the measurements of static or slowly changing parameters. However, measurements on dynamic process are beyond the scope of such instruments and therefore require special electronic devices.

9.1 Discharge

Measurement of flow discharge is a primary parameter for the model study. Laboratory instruments for measuring discharge consist principally of standard V notches and flumes for open channel flow.

The ultrasonic and electromagnetic flow meter devices require careful set-up and uniform flow approaching the meter location, and they should always be calibrated in place to verify accurate measurements.

The discharge measurement of any physical models can accurately be carried out with the standard V notches or ultrasonic or electromagnetic devices.

9.2 Water Level

Generally during the laboratory test, water surface elevation is measured with a point gauge over the water surface or with a hook gauge mounted in a stilling well to dampen fluctuations of the water surface. These gauges may be used to indicate differences in absolute elevations. Proper care should be taken for location of gauge so that intended measurements can be achieved with proper accuracy. In case of spillway, intake structures and bottom outlets, stage-discharge relationship has to be established for which observation of reservoir surface water level is of prime importance. Similarly the observation of down stream water level for different discharge is another important observation in headworks model study.

9.3 Velocity

One of the most important parameters to be measured accurately is the flow velocity. In case of headworks of RoR projects, almost every project consists the settling basins. The effective performance of this basin can be visualised with the flow velocity in it. Classic flow measurement devices, such as pitot tubes, current meters, and propeller meters, are still widely used, and for specialized applications researchers continue to adapt them for new situations.

Modern instruments, commonly used in the physical modeling technique, which can provide three-dimensional measurements of flow velocity are electromagnetic current meters. Other devices often used for the measurements of turbulence velocity are hot-wire and hot-film anemometers which are also suitable for air or water flow. Significantly sophisticated electromagnetic devices that are used in many hydraulic studies for velocity measurements are Laser-Doppler velocimeters (LDV) and Acoustic-Doppler velocimeters (ADV).

9.4 Pressure

Pressure is the another basic hydraulic parameter which is very important for the observation of the physical models of headworks. Normally in the spillway surface, chute and bottom outlets, pressure measurements are necessary so as to confirm the problem of cavitation and extent of erosion. The simplest method for measuring small positive water pressure is the manometer. Bourdon-tube pressure gauges are also widely used, but care must be taken in their selection as these gauges vary widely in their accuracy and precision.

With the advanced technology, varieties of transducers have been invented which can be used for the measurement of dynamic pressures. These transducers can be used for the measurement of positive, negative, differential, or absolute pressures, with a wide range of sensitivities, dynamic characteristics, and signal output options.

10. DESIGN OF MODELS

Any physical model has to be well designed and constructed as per the prevailing practices. Before the construction of models, the model drawings have to be prepared accurately to prevent errors in construction. These drawings have to be prepared containing sufficient information to enable the model makers to build a model which conforms to the designer's specifications.–In the same way, clear operating programme should be carefully planned. In general, the evaluation of model study consists of proving, by qualitative and quantitative tests to meet the design and operation requirements.

Success or failure in achieving desired results with least work and cost depends largely on the design of the model. Once the basic parameters affecting the problem have been identified, the first step is to select a model scale. One or more of the scaling criteria can be selected for the design of the model. The model should cover the enough area from far upstream to the downstream of the diversion structure so that the river behavior and sediment management can be well studied. Normally, the diversion dam/weir, intake structure, spillway, stilling basin, fish passes, settling basin etc are often subjected to free surface flow followed by Froude law. While bottom-outlet or diversion tunnel spillway should be modeled by Reynolds Law.

10.1 Data for Models

Various data has to be collected and compiled in a form of a report before the actual design of the model. The report may include the purpose of the headworks structure, hydrological and hydraulic calculation of the proposed design, structure layouts and their sizes, sediment data and past behavior of the river at headworks site supported by photographs, if available. In addition to these, survey data including the river profile and cross sections are very important for the design and construction of the model. Additionally, the data on the armoured bed material and boulders that are influencing the river hydraulic along the selected stretch is very important especially for steep gradient Himalayan River.

11. CONSTRUCTION OF MODELS

After the detail design of physical models the next step is to construct the model according to the planned operating programme. The planning of stages of testing programme has great importance to the construction of models.

A well-equipped workshop and experienced craftsperson are crucial to the successful construction of most physical models. Since it is necessary to obtain greater accuracy in building a model, care should be taken to obtain the required tolerances, particularly in the critical parts. Craftsperson must be capable of constructing models to tight tolerances, at areas having rapid changes in direction of flow and with high velocities.

In case of river model construction, the model shall construct covering reasonable distance at up stream and down stream stretch.

11.1 Establishment of Control Points

Headworks of hydropower projects can be built in a small area compared to river and large water bodies. In physical models, horizontal and vertical control points are established using similar methods as used in the full-scale construction works. Benchmarks for the model structures should be established externally. The location shall be selected from easy access point of view during and after the model construction. The instruments used for establishing control points are also similar to full scale constructions, which are EDM, Theodolite, Levelling Machines etc.

Photo 1: Upper Tama Koshi Hydropower Project: view of river stretch selected for headworks site (left) and 1:60 scale model at Hydro Lab Pvt. Ltd. (Courtesy: Hydro Lab Pvt. Ltd.)

11.2 Elements of Model

The elements of a model vary with the nature of the prototype structures. Instrumentations are essential part of every model and the type of instruments may differ as per the nature of the model.

The major elements of headworks of the hydropower projects mainly include upstream river portion, diversion structures, intake structure, spillway, undersluice, stilling basin, downstream river portion, gravel trap, approach canal/conduit and settling basins.

11.2.1 Headworks Area

The diversion structure may be of conventional weirs, gravity dam or gravity structure armoured with big boulders etc. The main elements of headworks area are built to reproduce the similar effect as in the prototype. In case of RoR and PRoR, the model essential consists of river stretch at upstream and downstream of diversion structure. The water coming from the measuring flumes has to be directed towards the river stretch. The flow has to be uniformly distributed to the river channel so that the flow regime can be obtained in the model similar to the prototype. Proper attention has to be paid to stabilize the flow in the inlet of river channel by providing baffle system as shown in Photo 2.

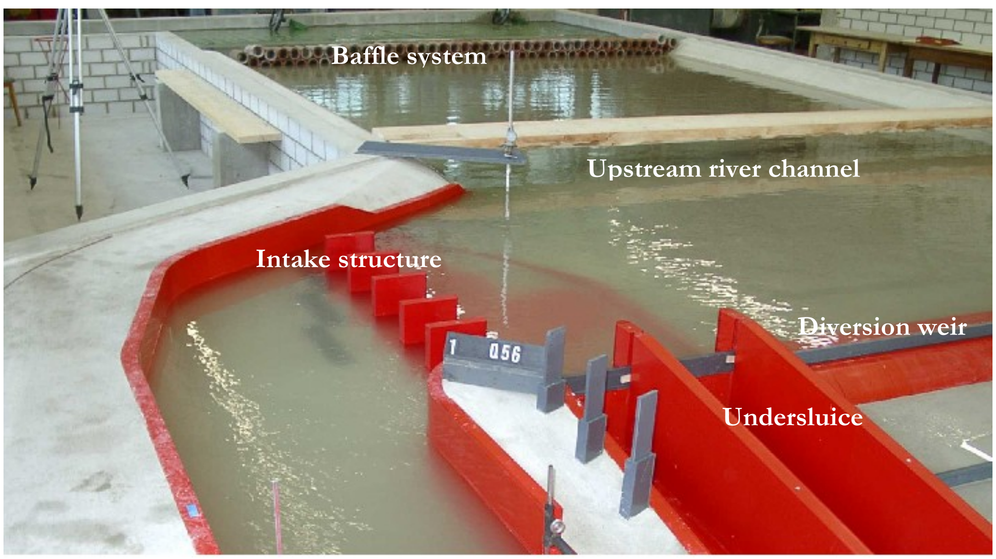

Photo 2: View of physical model (scale 1:20) of headworks of Eisenbahnerwehr RoR project (Augsburg, Germany) at VAO, TUM (Bhattarai, 2003)

Similar to upstream of diversion structure, the headworks elements such as diversion weir/ dam, spillway, undersluice, intake structures, settling basin and downstream of diversion structure has to be modeled to reproduce designed hydraulic phenomena. Sand or gravel is often placed in the downstream to aid in the study of the scour patterns and designs are judged to some extent by their scouring tendencies.

11.2.2 Instrumentation

The importance of proper instruments installation and their use is strongly emphasized for comparison of measurement. Provision should be made for suitable instrumentation while the model is in the design stage. Sufficient number of instruments has to be installed at the proper location for defining the complete local action.

11.3 Construction Materials

The materials for model construction can differ for various structures of the same model. In the similar manner, a wide variety of materials may be suitable or desirable for model construction depending upon the cost of materials and anticipated investment in a model. The preferred materials and methods of construction can also be different from one laboratory to another. The major concerns during the selection of material should be such that it should provide the similar surface in the scaled model as in prototype. On the other hand, the downstream of spillway or scour protection area can be protected with appropriate material as required.

11.3.1 Upstream of Diversion Structure

The headworks model requires the construction of upstream topography, especially in the run of the river hydropower projects in the steep gradient rivers. This can be accomplished using many different materials. Fixed topography can be constructed from concrete, foam, or any other durable material. For this, at first there shall be produced cross sections of plywood as templates in a reasonable interval. A layer of concrete placement may be required for the installation of the templates. Flow surfaces can be trowelled smooth or roughened, as needs dictate to resemble the scaled model of a prototype. The bathymetry between templates might be infilled with coarse sand or fine gravel, and only the focal area of flow interest fixed with mortar to make it water tight. Suitable elevated walkways shall be needed to provide access to various parts of the model.

11.3.2 Headworks Structures

The headworks structures such as weir/dam, abutments, piers, chute, settling basin, bottom outlets and stilling basin can be made of either PVC materials or epoxy coated plywood or acrylic plastic (Lucite or Plexiglas). Epoxy-coated, marine-grade plywood produces a durable product in water and is smooth enough to be used for planar flow surfaces. As a low cost technology the enamel painted plywood can also be used. However, bricks, stone masonry, concrete blocks etc are also used to support and stabilize structures in the model.

11.3.3 Gates

Gates of the smaller sizes may be of machine fabricated plastic materials. Since corrosion affects the performance of the model, while using the metal parts for such cases, the main goal should be to avoid the corrosion as far as possible. Therefore metal parts used for the model construction should always be of aluminium, brass, PVC material or stainless steel. One critical requirement of the materials used in construction of hydraulic models is that they be durable and resistant to dimensional changes caused by the absorption of water.

11.3.4 Materials for sediment simulation

Bed load transport by the river and its flushing arrangement through the diversion structure are often studied in the model. Bed load material can be scaled down to the gravel or sand size for boulders and large size gravel. Another difficult parts of the physical model study of headworks is the study of settling basin. The trapping efficiency and flushing pattern together with its flushing capacity of flushing gallery have to be visualised in the model study. Since it is difficult to scale down sand, silt or other suspended particles in a model, some other measures have to be adopted. As a practice, walnut shell dust, saw dust or coal powders are used as fine sand. The mean diameter of natural silt and fine sand shall be scaled down in proportion to the density or fall velocity. Due to the advanced technology, there are some manufacturers who make artificial sediments ranging in size from 0.030 - 4 mm, which can be used in fine sediment simulation. Special sediment feeding apparatus has to be built for supplying a uniform and continuous supply of sediments during the model tests.

12. OPERATION OF MODEL

Any physical model testing consists of systematically examining each proposed design with regard to possibilities for design improvement, reduction in cost of construction and maintenance for the prototype. Therefore any physical model set up and model operation should completely follow the design consideration as foreseen. It is highly recommended that the designer should visit the laboratory frequently for the observation of the model construction and operation. This would be helpful to the designer to change the design parameter as required.

12.1 Initial Adjustments

These adjustments include calibration of instrumentation and other measuring devices used for the observation of model parameters. The observation of pumping system and/or constant head tank, water leakage from model components, accessibility in and around the model instrumentation and over all flow situations also come under the initial adjustments. The correct functioning of all instrumentation and water tight situation of the model completes the initial adjustment.

12.2 Model Verification and Validation

Model verification activities require tuning of the model, to ensure that it satisfactorily reflects the performance of the prototype. During the model verification, repeated trials are performed in the model to reproduce a single or series of known events. In a physical model, built for one particular case study of headworks of hydropower project, the known event may be a combination of water level and discharge, and the adjustments will include small modifications of boundary conditions. A good analogy for physical model calibration and verification could be the use of numerical modeling.

The model verification ensures that the model behaves properly as anticipated based on the mathematical relations. The model verification also reveals that there are no mathematical errors or other inconsistencies commonly referred to as bugs. Verifying a hydraulic model consists of ensuring that the model obeys known physical laws.

In the similar manner, the model validation confirms that the model reproduces the relevant physical processes that affect the performance of the structure or device being tested. Having completed the initial adjustments, verification and validation of a model, the modeler can also have the maximum confidence to the results obtained from the model observation.

12.3 Model Observation and Modification

The most important and final activity of a physical modeling is the testing and modification along with data recordings of the model of a particular structure to identify the flow processes and prove a design concept.

12.3.1 Diversion Structures

The primary diversion structure of the hydropower projects are un-gated weir/spillway or gated spillway and dam. From the physical models of these structures, the optimum length of the weir/spillway and abutment heights for design floods can be finalized. In a similar way, the adjustment of the divide wall upstream of the weir can be determined for efficient flow diversion towards the intake structure. The main objective of the model study of weir or spillway is to determine the discharge coefficients and pressure at the crest for different shapes. In case of pressure measurements, the profile of the weir or spillway shall be tested for the existence of sub-atmospheric pressures. For most spillways, it is desirable to have positive pressures on the spillway face for all cases except for maximum design flood which does not suppose to appear frequently.

12.3.2 Intake Structure

The model testing for discharge capacity of intake structure can be different depending upon the type of intake structure. If the intake structure is of overflow weir type, similar observations as mentioned in Section 12.3.1 are adopted. Whereas, if the intake structure is of orifice type, the discharge coefficient can be determined for different gate openings for known value of measured discharge, crest lengths and upstream and downstream water levels. The model can be modified for different crest length and orifice opening. The model shall be observed and modified if required for the appropriate function of the intake structure for handling the floating debris, excluding the bed loads, abstract enough water during low flow and safe operation during floods.

12.3.3 Stilling Basin

In the design of stilling basins with free jets, it is important to know the position of the point of downstream impact. The model study can reveal the height and location of impingement of jumps. The problem of erosion that occurs in the downstream of spillway, stilling basin, flushing sluice can also be studied in model using the different size of gravels that are scaled down from prototype riprap using the Froude Law. The size of downstream protection blocks and extent of their location can be modified to achieve the safe and optimum design.

12.3.4 Settling Basin

In case of settling basin modeling, the surface velocity measurement is of utmost importance. From the theoretical relations, the trap efficiency of the basin can be determined for the observed surface velocity. The next part is the observation of settling velocity of sediments. Special type of artificial sands or other materials (refer Section 11.3.4) downsized for the use in physical modeling, can be supplied at the entrance of settling basing and their settlements can be visualised. The recording of settling process and pattern can be done with special still/video camera. The flow pattern at the entrance and exit of settling basin, amount of turbulence, and uniformity of flow in vertical and horizontal direction throughout the settling basin can also be observed from the model by using the dye performance. Another aspect of the settling basin modeling is to check the flushing capacity and pattern of the flushing galleries. The bottom shape (hopper or rectangular) of the settling basin can be varied to achieve the maximum flushing capacity.

12.4 Model Data Recording

Both still photographs and videotapes are an indispensable part of the most model studies. These visual records provide documentation of model configurations, test equipment arrangements, and flow conditions. These recordings can be used as a part of final reports and documents of the model study. In addition to documenting all phases of the model study, photographs and videotapes may also serve as a primary means for recording model data during testing.

The most important part of any physical modeling for their best results is the accuracy of data observation and recording. Normally flow parameters, such as discharge, water level, erosion depths etc are manually observed. However, these flow parameters can also be recorded electronically. The rapid growth in computerized data-acquisition system made the possibility of computer-aided measurement and computer-control capabilities. This makes the model researcher efficient to collect data and observe the operation of the model.

12.5 Supervision and Maintenance of Model

The supervision and maintenance of a physical model is a routine job. During the process of model observation, many modifications may require such as dismantling of some part of the model and re-construction. These works should be carried out under great attention to any leakage or distortion in the model. The model and instrumentations may also require regular maintenance. The regular supervision and maintenance of the model not only helps to keep model in proper condition but also ensures the precision of the observed results.

13. NUMERICAL MODELING

There are some problems in the hydraulics of civil engineering works that require very bulky physical models and tedious observation process if investigated by the physical modeling. On the other hand, in the recent years, several computer softwares has been developed for the easy and fast numerical calculations with respect to time and space for such problems. In case of hydropower plants, linear problems such as surges in tunnel systems and backwater levels upstream of dams were the first in which numerical modeling was used.

13.1 Philosophy of Numerical Modeling

Fluid flow models that are solved numerically can be categorized by the appropriate governing partial differential equations as shown in Table 2. For the majority of hydraulic applications involving water flow, the numerical models are set up for solving a steady-state incompressible flow in the form of the Navier-Stokes equations. The velocities and pressures in the domain will be solved in the computation for each time step in a transient analysis. The partial differential equation can be difficult to solve because of the inherent non-linearity, second order form, four-dimensional, and inter-related variables.

Table 2: Different flow models and their governing equations

Flow model	Governing equation
Ideal, irrotational and incompressible	Euler's momentum equation, and conservation of mass
Viscous, incompressible without inertia effects	Stoke's flow or creeping
Viscous, incompressible with inertia effects	Navier-Stokes equation
Viscous, compressible with inertia effects	Navier-Stokes equation with compressibility terms

The computation becomes more complex when a turbulence model is introduced in the equation. It should be noted that while the Navier-Stokes equations can be solved exactly for many classical problems, any attempt to model the effects of turbulence could only be done in a statistical approximation manner. The Reynolds-averaged Navier-Stokes equations are commonly used. This is an expanded form of the Navier-Stokes equations that carries the Reynolds stresses terms.

The details and different numerical methods of solving the above-mentioned equations can be found in standard Computational Fluid Dynamics (CFD) textbooks. Typically, the CFD modeling process can be carried out in the following steps:

Select the appropriate governing equations - some idea of the Reynolds number would provide a good starting point.

a. Create the flow domain and obstacles that represent no flow regions such as spillway and piers. An appropriate grid or mesh will then be generated.
b. Define the boundary and initial conditions - for example, upstream head and tail-water depth.
c. Select the fluid properties, wall roughness and turbulence model.
d. The program will solve for the unknown variables such as pressure and velocity with time at the grid points or at the cell/mesh center.
e. Post-process results to extract the desired information for design.

Besides solving for pressure and velocity in the flow domain, one important aspect in modeling open-channel flow is the accurate tracking of the free surface. Sometimes, multiple free surfaces would be involved in the model, for example, a stream of water entering a plunge pool after leaving a flip-bucket. A well-known computational technique implemented in the CFD code, is FLOW-3D, which is widely used for the spillway analyses. The program solves the Navier-Stokes equation by the finite difference method.

Design of run- of river water intakes is a particular problem in sediment carrying rivers. Often a physical model is done to assist in hydraulic design. However a numerical model called Sediment Simulation In Intakes with Multi-Block Option (SSIIM) can be used for verifying the physical model studies. The SSIIM model has been used to compute trap efficiency and model bed changes in a sand trap.

If numerical modeling softwares are available, it is wise to use the numerical modeling results together with physical modeling to verify the design and optimization of headworks of hydropower project.

PART 3 – CONSTRUCTION

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Part

3

Construction

1. PURPOSE OF GUIDELINES

Part 3 of the *Design Guidelines for Headworks of Hydropower Projects* provides procedural requirements for the construction of headworks of run-of-river hydropower projects. The guidelines are intended to ensure that the headworks are constructed in accordance with the design intent.

2. SCOPE OF GUIDELINES

The guidelines lay down the basic considerations to be followed for construction of headworks of run-of-river projects. They provide guidance on construction preparation, construction layout, river diversion, foundation mapping and treatment, concreting, instrumentation and monitoring as well as documentation requirements and procedures.

3. COORDINATION BETWEEN DESIGN AND CONSTRUCTION

During the design and construction periods, close coordination between design and construction personnel is necessary to:

- thoroughly orient the construction personnel with the project design intent;
- ensure that new field information acquired during construction is assimilated into the design, and
- ensure that the project is constructed according to the intent of the design.

Suitable mechanisms for achieving these objectives are discussed below.

3.1 Orientation of Construction Personnel

Before construction commences, the project designers shall orient the construction personnel with the design philosophies, assumptions regarding site conditions and function of project structures and the intent of the technical provisions in the plans and specifications. The design personnel shall also prepare a report to aid the construction engineers in supervision and inspection of the construction works.

3.2 Incorporation of Field Information into Design

The design personnel should visit the construction sites to ensure that:

- site conditions throughout the construction period are in conformance with design assumptions and principles as well as contract plans and specifications;
- project personnel are given assistance in adapting project designs to actual site conditions revealed during construction, and
- any engineering problems not fully assessed in the original design are observed, evaluated and acted upon appropriately.

Such site visits shall specifically be made at the following construction milestones of the project:

- excavation of cutoff trenches, foundations, and abutments for dams and appurtenant structures;
- excavation of borrow areas and placement of embankment dam materials early in the construction period, and
- observation of field conditions that are significantly different from those assumed during design.

3.3 Attainment of Design Intent

In order that the construction works do not deviate from the intent of the design, any design modification or alternative proposed by the construction personnel or the contractor shall be evaluated by the design personnel prior to its implementation. The design personnel shall evaluate the technical adequacy of the proposal and ensure that structural safety of the project component is not compromised.

4. CONSTRUCTION LAYOUT

Construction surveys shall generally be performed to meet Third Order-Class II (1:5,000) accuracy. Some stakeout work for earthwork clearing and grading, and other purposes, may need only be performed to meet Fourth Order (1:2,500-1:20,000) accuracy requirements. Other stakeout work, such as bridge pier alignment, may require Second Order (1:20,000-1:50,000) or higher accuracy criteria. Construction survey procedural specifications shall follow requirements of Part 1A of the guidelines.

5. RIVER DIVERSION

The design for a dam which is to be constructed across a stream channel must provide diversion of the streamflow around or through the dam site during the construction period. The extent of the diversion problem will vary with the size and flood potential of the stream. The following factors shall be considered in a study to determine the best diversion scheme:

1. Characteristics of streamflow.
2. Size and frequency of diversion flood.
3. Methods of diversion.

5.1 Characteristics of Streamflow

Depending upon the size of the drainage area and its geographical location, floods on a stream may be the result of snowmelt, seasonal rains or cloudbursts. Because each of these types of runoff have their peak flows and their periods of low flow at different times of the year, the nature of runoff will influence the selection of the diversion scheme. A site subject only to snowmelt floods will not have to be provided with elaborate measures for use later in the construction season. A site where seasonal rains may occur will require only the minimum of diversion provisions for the rest of the year. A stream subject to cloudbursts which may occur at any time is the most unpredictable and probably will require the most elaborate diversion scheme to handle both the low flows and flood-flows at all times during the construction period.

5.2 Selection of Diversion Flood

In selecting the flood to be used in the diversion designs, consideration shall be to the following:

1. Duration of construction to determine the number of flood seasons which will be encountered.
2. Cost of possible damage to work completed or still under construction if it is flooded.
3. Cost of delay to completion of the work, including the cost of forcing the contractor's equipment to remain idle, while the flood damage is being repaired.
4. Safety of workmen and possibly the safety of downstream inhabitants in case the failure of diversion works results in unnatural flooding.

After an analysis of these factors is made, the cost of increasing the protective works to handle progressively larger floods can be compared to the cost of damages resulting if such floods occurred without the increased protective work. Judgment can then be used in determining the amount of risk that is warranted.

5.3 Methods of Diversion

The method or scheme of diverting floods during construction depends on:

a. Magnitude of the flood to be diverted.
b. Physical characteristics of the site.
c. Type of dam to be constructed.
d. Nature of the appurtenant works such as the spillway, penstocks.
e. Probable sequence of construction operations.

The objective is to select the optimum scheme considering practicability, cost, and the involved. The diversion works should be such that they may be incorporated into the overall construction program with a minimum of loss, or delay.

Construction diversion may be achieved using one or a combination of the following methods:

a. Tunnels driven through the abutments.
b. Conduits through or under the dam.
c. Temporary channels through the dam.
d. Multiple-stage diversion over the tops of alternate construction block of a concrete dam.
e. Coffer dams.

5.3.1 Tunnels

In a narrow canyon, a tunnel may prove the most feasible means for diversion, either for a concrete dam or for earthfill dam. The stream flow may be bypassed around the construction area through tunnels in one or both abutments.

The advisability of lining the diversion tunnel will be influenced by the cost of a lined tunnel compared with that of a larger unlined tunnel of equal carrying capacity; the nature of the rock in the tunnel, as to whether it can stand unsupported and unprotected during the passage of the diversion flows; and the permeability of the material through which the tunnel is carried, as it will affect the amount of leakage through or around the abutment.

Some means of shuttling off diversion flows must be provided; in addition, some means of regulating the flow through the diversion tunnel may be necessary. Closure devices may consist of a timber, concrete, or steel bulkhead gate; a slide gate; or stoplogs.

5.3.2 Conduits

The outlet works for an earthfill dam often entails the construction of a conduit which may be utilized for diversion during construction of the dam. This method for handling the diversion flows is an economical one, especially if the size of the conduit required for the outlet works is adequate to carry the diversion flows. Where diversion requirements exceed the capacity of the completed outlet works, an increase in capacity can be obtained by delaying the installation of gates, valves, pipe, and trashracks until the need for diversion is over. Increased capacity also can be obtained by increasing the height of the cofferdam, thereby increasing the head.

5.3.3 Multiple-Stage Diversion for Concrete Dams

The multiple-stage method of diversion over the tops of alternate low construction block or through diversion conduits in a concrete dam requires shifting of the cofferdam from one side of the river to the other during construction. During the first stage, the flow is restricted to one portion of the stream channel while the dam is constructed to a safe elevation in the remainder of the channel. In the second stage, the cofferdam is shifted and the stream is carried over the low block through diversion conduits in the constructed section of the dam while work proceeds on the unconstructed portion of the dam. The dam is then carried to its ultimate height, with diversion ultimately being made through the spillway, penstock, or permanent outlets.

5.3.4 Cofferdams

A cofferdam is a temporary dam or barrier used to divert the stream or to enclose an area during construction. The design of an adequate cofferdam involves the problem of construction economics. Where the construction is timed so that the foundation work can be executed during the low water season, use of cofferdams can be held to a minimum. Where the streamflow characteristics are such that this is not practicable, the cofferdam must be so designed that it is not only safe, but also of the optimum height. The height to which a cofferdam should be constructed may involve an economic study of cofferdam height versus diversion works capacity, including routing studies of the diversion design flood, especially when the outlet works requirements are small.

Generally, cofferdams are constructed of materials available at the site. The two types normally used in the construction of dams are earthfill cofferdams and rockfill cofferdams, the design considerations of which closely follow those for permanent small dams of the same type. Other types not as common include timber or concrete cribs filled with earth or rock, and cellular steel cofferdams filled with earth or rock.

5.3.5 Construction Diversion Flood

The nature of runoff and construction program will influence the selection of the diversion scheme. Stream flood data for non-monsoon period shall be appended for as many years as possible. In selecting the design flood for diversion, consideration shall be given to the following:

a. Time and duration of the construction work to consider the number of flood seasons, which will be encountered.
b. The cost of possible damages to work completed or still to be completed if it is flooded.

c. The cost of delay to the completion of the work including the cost of forcing construction labor and equipment to remain idle while the flood damage is being repaired.

d. The safety of downstream inhabitants in case of failure of diversion works resulting in unnatural flood.

Cofferdams are temporary structures required for diverting the river flow from the main work areas during the period of construction of permanent structures. The design flood for cofferdams may be determined based on floods of 5 to 25 year frequency depending on the period during which they required to be in operation and the risk of damage involved in their failure.

6. FOUNDATION MAPPING

Geological mapping will be made for all rock and soil area exposed after excavation for the foundation and slopes around civil structures. In these studies, the geological mapping will be carried out in more detail especially in critical areas of the civil structure locations at the scale of 1:100 or 1:200 or as required. Important geological data should be collected related to:

- Overburden type and thickness
- Rock types
- Geological contact between different rock units
- Dip and strike of bed rock (orientation)
- Dip and strike of major joints, faults.
- Landslide, rock slides
- Detailed mapping of excavation pits (foundation)
- Major and minor thrusts, faults and folds
- Water seepage
- Unstable slopes
- Preparation of as built geological plans and sections for different civil structures
- Detailed joint mapping in rock-outcrops, etc.

As all slopes and foundations will be covered either by concrete or other backfill material, mapping of the exposed part will be necessary for record purpose. At some civil structure locations, there may be a need of new investigation like exploratory drilling, rock and soil mechanical tests, permeability, grouting test etc. during construction.

The person in charge of foundation mapping should be familiar with design intent via careful examination of design memoranda and discussion with design personnel. The actual geology should be compared with the geologic model developed during the design phase to evaluate whether or not there are any significant differences and how these differences may affect structural integrity. The person in charge of foundation mapping should be involved in all decisions regarding foundation modifications or additional foundation treatment considered advisable based on conditions observed after preliminary cleanup. Design personnel should be consulted during excavation work whenever differences between the actual geology and the design phase geological model require clarification or change in foundation design. Mapping records should include details of all foundation modifications and treatment performed.

7. FOUNDATION TREATMENT

Foundation treatment will be carried out beneath the diversion dam and other large structures in order to provide adequate support for all loading conditions, and in the case of the diversion dam, to reduce filtration. Consolidation grouting should be done only if found necessary when the foundation is exposed and inspected.

Any rock material within the dam foundation having strength or deformation characteristics inferior to that expected for founding the dam should be removed. Following removal of overburden material, objectionable overhanging and protrusions should be cut. All depressions and major joint openings under the dam foundation should be filled with dental concrete and grouted as required.

The grout curtain under the dam, if necessary should be constructed from a gallery in the diversion structure. The drilling should be performed down to required depth at selected interval and orientation. The grout curtain should be extended in the vicinity as required.

All grout holes should be pressure tested. All grout holes should be grouted to refusal with cement grout, Blaine 4000 or finer. If water leakage is more than designed quantity then new grout holes should be drilled at split spacing, pressure tested and grouted. The process should continue until all grout holes show leakage less than expected for the area and/or cement consumption becomes less than 35 kg/m. The closing criteria should be found out based on geology and grout take relations.

Pressure applied for consolidation and curtain grouting should be site specific depending upon rock conditions.

8. CONCRETING

8.1 Construction Materials

The design of concrete dams involves consideration of various construction materials during the investigations phase. An assessment is required on the availability and suitability of the materials needed to manufacture concrete qualities meeting the structural and durability requirements, and of adequate quantities for the volume of concrete in the dam and appurtenant structures. Construction materials include fine and coarse aggregates, cementitious materials, water for washing aggregates, mixing, curing of concrete, and chemical admixtures. One of the most important factors in determining the quality and economy of the concrete is the selection of suitable sources of aggregate. In the construction of concrete dams, it is important that the source have the capability of producing adequate quantitives for the economical production of mass concrete. The use of large aggregates in concrete reduces the cement content.

Pozzolanas may be used with advantage in cement concrete mix with a view to effecting economy in construction and to control alkali aggregate reaction. The pozzolana cement concrete is considered to have less resistance against abrasive forces and cavitation. The use of such concrete should, therefore, be avoided for the top layer of crest and downstream glacis portions, where ordinary Portland cement concrete is recommended.

The concrete which comes immediately in contact with trunion girder and takes the thrust in bearing from it should be of non-shrinkage quality for a minimum thickness of 300 mm.

8.2 Temperature Control

For control of concrete cracking and development of high thermal stresses, temperature control of concrete is necessary. This may be achieved by precooling of concrete in gradients so as to obtain desirable placement temperature of concrete. The concrete, as deposited, should have a temperature of not more than the stipulated value (usually 15 to 21°C for spillway concrete in hot climate). The temperature of the concrete should be not less than 5°C in moderate weather or 10°C when the mean daily temperature is lower than 5°C. As an additional precaution when the mean daily temperature is lower than 5°C, one percent calcium chloride by mass of cement may be used to bring the concrete to a stage of greater maturity at the end of the specified period of protection.

8.2.1 Methods of Temperature Control

Most commonly used methods are precooling, post cooling and reducing heat of hydration by proper mix design. The ideal condition would be simply to place the concrete at stable temperature of dam and heat of hydration removed, as it is generated, so that temperature of concrete is not allowed to rise above stable temperature. However this is not possible to achieve practically. Therefore, the most practical method is to precool concrete so as to restrain the net temperature rise to acceptable levels.

8.2.2 Precooling

One of the most effective and positive temperature control measure is precooling which reduces the placement temperature of concrete. The method, or combination of methods, used to reduce concrete placement temperatures will vary with the degree of cooling required and the equipment available with the project authority or the contractor. In this method usually the fine and coarse aggregates and the water are separately cooled to the requisite temperatures.

8.2.3 Post Cooling

Post cooling is a means of crack control. Control of concrete temperature may be effectively accomplished by circulating cold water through thin walled pipes embedded in concrete. This will reduce the temperature of newly placed concrete by several degrees, but the primary purpose of the system is to accelerate the subsequent heat removal and accompanying volume decrease, during early ages when the elastic modulus is relatively low. Post cooling is also used where longitudinal contraction joints are provided in order to reduce the temperature of concrete to the desired value prior to grouting of transverse contraction joint. Post cooling will create a flatter temperature gradient between the warm concrete and the cooler exterior atmosphere which, in turn, helps in avoiding temperature cracks. Other methods such as evaporative cooling with a fine water spray, cold water curing and shading may prove beneficial, but the results are variable and do not significantly affect the temperature in the interior of massive placement.

8.2.4 Pre-design Measures for Achieving Temperature Control

Cracking tendencies in concrete due to temperature changes may be reduced to an acceptable level by suitable design and construction procedures. The volumetric changes are caused by the temperature drop from the peak temperature attained by concrete shortly after placement to the final stable temperature of the structure. To bring down net temperature rise to acceptable limits, in order to avoid cracking, the parameters given below are to be predetermined or suitably modified for a concrete dam.

1. The cement content in the concrete plays an important role in evolution of heat of hydration. By suitably modifying the concrete mix design, the quantity of cement per cubic metre of concrete may be reduced so as to reduce the amount of heat generated by cement.
2. The height, of the placement lift is generally governed by economic considerations. Shallow lifts not only slow down the construction but also result in increased construction joints which entail additional costs for cleaning and preparation for placement of the next lift. The thickness of lift is also related to the temperature control measures proposed and the ambient temperature at the site. To reduce the net temperature rise, the lift thickness should be reduced if time is not a constraint.
3. There is always some time lag between placement of concrete in successive lifts. Depending upon ambient temperatures, these delays can be beneficial or harmful.

Allowance of sufficient time between two construction lifts to allow large dissipation of heat from the surface is an effective and important factor to control the rise in the temperature during construction. The minimum elapsed time between placing of successive lifts in any block is usually restricted to 72 hours, but temperature studies should be made to relate heat loss or heat gain to the placement lift heights.

4. During summer months the ambient temperature is very high and, as such, heat gain from the atmosphere is also high. This may be reduced by continuous curing of the concrete by sprinklers using river water, after concrete has set. In such a case, ambient temperature for calculation purposes may be assumed as average of curing water temperature and actual ambient temperature.

8.2.5 Tolerances

All concrete structures should be constructed to exact lines, grades and dimensions established. However, inadvertent variations from the established lines, grades and dimensions should be permitted to within tolerable limits.

9. STRUCTURAL SAFETY

The construction phase serves as an important link in attaining structural safety in the project. It is during this phase that the structural safety concepts envisaged in the design are implemented, often with suitable modifications to ensure that the final design is compatible with the actual site conditions. The operational success of the project and the ease of its maintenance are also directly related to the care and competence exercised during the construction phase.

Structural safety issues shall be addressed during the construction phase by incorporating the following measures in the project development plans:

- proper coordination between design and construction, and
- suitable means for control and inspection.

The first of the two measures shall be achieved through the means discussed in Section 3. The second measure shall be implemented by exercising necessary caution during construction, preparing appropriate construction plans and specifications and by conducting quality control programs, instrumentation and monitoring as well as periodic construction inspections.

9.1 Precautions During Construction

Structural safety of project components shall be enhanced by guarding against typical problems during construction. Some such problems are:

- differential settlement and voids or cracks within embankments due to dissimilar compressibility at contacts between fill and rock or concrete;
- fill and rock/concrete contact along critical areas at rock abutments, spillway contacts, adjacent to conduits and at contacts with existing fill which have had time to undergo settlement;
- placement and compaction of fill or concrete complicated because of restricted access;
- initial fill placement on rock foundation;
- differential settlement and cracking as well as reduction in stability of dam and foundation due to failure in detecting and removing localized areas of soft foundation material.
- failure to achieve an adequate cutoff to sound rock or an impervious strata;
- failure to achieve grouting status as envisaged in design;
- failure of backfill grouting in tunnel lining, etc.;

- failure to achieve adequate reduction in seepage volume or pressure with grout curtain or to seal off large channels or voids;
- failure to achieve adequate surface treatment of joints, potholes, treatment of overhangs, loose rock on abutments or inadequate bond between embankment and foundation.
- piping because of failure to achieve filter criteria due to segregation of material which may occur in transition to rockfill or in exit details;
- inadequate drainage capacity because of improper gradation, dirty materials, material breakdown during placement or contamination of the filter with impervious material from the adjacent core due to traffic or due to surface erosion caused by rainfall;
- failure to detect critical zones such as void or strata, or lenses, of pervious materials in foundations without cutoffs.

9.2 Construction Plans and Specifications

For each project to be constructed, a set of construction plans and specifications shall be prior to start of construction. These plans and specifications shall contain adequate information on the technical features of the project to form the basis for project inspection and control. The specifications shall include provisions that require the contractor to be responsible for various types of controls that are consistent with the requirements of the project license. Adherence to the plans, specifications and other control measures shall be verified by the Quality Control Inspection Program discussed in Section 9.3.

9.3 Construction Quality Control Program

Prior to construction, a quality control inspection plan (QCIP) shall be prepared considering the design assumptions and requirements. The QCIP shall describe in detail procedures for material testing, record keeping, report submittals and field inspection for quality control during construction. It shall also provide details for monitoring of erosion control and other measures that would be required for the protection of the environmental integrity of streams and other areas affected during construction.

9.4 Instrumentation and Monitoring

An instrumentation and monitoring program shall be conducted during the construction of major projects that include extensive cofferdam arrangements or other large impounding structures, construction of new dams, spillways or embankment structures and the rehabilitation or modification of existing project features. The program shall be properly designed to assure that the project facilities are appropriately monitored during project construction.

9.4.1 Instrumentation and Monitoring Plan

Before the start of construction, a detailed pan of the types of monitoring devices, their location and frequency of readings shall be prepared. The plan shall be carefully developed taking into consideration the type of measurement required and time in which the information is desired. The planning process for an instrumentation program shall include an evaluation of the adjacent terrain as well as the structure and shall identify sufficient time and resources to obtain background or baseline data for each type of measurement desired. Other considerations include the risk of damage during construction, effects of a severe environment on the instruments, maintenance and personnel requirements for data collection and evaluation.

9.4.2 Data Reduction and Evaluation

The instrumentation data shall be reduced and interpreted in a timely manner to ensure a responsive safety evaluation of the project. For all projects, this reduction, interpretation and evaluation shall occur as soon as conditions warrant from the time that the data were obtained. The evaluation of the data shall follow immediately. As a minimum, all data shall be plotted as instrument response with respect to time as well as reservoir level or other range of loading.

10. POST CONSTRUCTION REQUIREMENTS

After the completion of construction, various aspects of the construction works shall be documented for future reference. The document shall include field control data on methods of compaction, in-place unit weight and moisture content, piezometers, surface monuments and slope indicators for use in operation and maintenance of the project. The licensee shall also prepare plans showing the work as actually constructed and prepare the as-built drawings.

PART 4 –OPERATION AND MAINTENANCE

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4

Operation and Maintenance

1. PURPOSE OF GUIDELINES

Part 4 of the *Design Guidelines for Headworks of Hydropower Projects* provides procedural guidance for the operation and maintenance of headworks of run-of-river hydropower projects in Nepal. The guidelines are intended to ensure that the headworks are operated and maintained in accordance with standard practices to satisfy the design intent of the various components of the headworks.

2. SCOPE OF GUIDELINES

The guidelines lay down the basic operation and maintenance procedures to be followed at headworks of run-of-river projects to ensure that the headworks components perform as designed. They also provide guidance on documentation requirements and procedures on headworks performance, hydrology and sediment, etc.

3. SCOPE OF OPERATION AND MAINTENANCE

Operation and maintenance of the headworks of a run-of-river hydropower project shall consist of the following activities to ensure that the headworks components perform as intended:

a. Proper planning to ensure that design assumptions and considerations are not violated during operation.
b. Continuous monitoring and performance evaluation / prediction of headworks components.
c. Adoption of remedial measures, where necessary, to accommodate unforeseen conditions.
d. Proper maintenance of headworks components.

These activities shall be performed by adopting the procedures discussed in the following sections.

4. PREPARATIONS FOR OPERATION AND MAINTENANCE

Before the headworks structures are put into regular operation, various activities related to their operation and maintenance shall be properly defined and organized. To this end, the activities listed in the following sections shall be performed prior to the commencement of the operation of the headworks.

4.1 Project Completion Report

A project completion report containing all design parameters and detailed as-built drawings of structures, equipment and accessories shall be prepared. This report shall always be available at the project site as ready reference to the plant operators.

4.2 Operation and Maintenance Manual

A detailed project operation and maintenance manual containing technical details and effective rules for good housekeeping shall be prepared during the construction phase of the headworks. The manual shall provide guidance and instructions to project personnel for proper operation and maintenance of the facility. For this purpose, the manual shall include the following topics:

- Critical features of the dam including design features with safety limits.
- Probable failure modes that could lead to structure failure.
- A history of problems and how they could adversely affect the structure under stress.

4.3 Training of Operating Personnel

The headworks operators shall be properly trained for normal operation and emergencies. During the training, they shall be provided with a complete description and a set of plans of the headworks. Each operator shall also be given complete instructions and information on the following topics:

- Operation of gates and other equipment so that the operator is prepared for emergencies that may arise.
- Detailed emergency instructions, including those for floods.
- All types of interruptions that can be foreseen.

The operators shall be trained to perform the following duties:

- Ensure proper flow of water from headworks to the power plant.
- Ensure that the trash rack is clean and other parts of the headworks are trouble free to allow unhindered flow of water to power plant.
- Raise and lower crest gates and control river level according to the demand of power system.

In addition, the operators shall make periodic inspections and reports on the general condition of the headworks, particularly where settlements, slides, leakages, sloughing of banks, scour, washouts and other sources of trouble may develop, especially during monsoon season. These inspection reports by the operators shall be largely for calling immediate attention to impending difficulties that may require a more detailed study. These reports shall be sufficiently detailed to ensure that the operators are vigilant.

5. PERFORMANCE PREDICTION

During the initial project design, the physical properties of the construction materials, design data, loading conditions and the appropriate factors of safety shall be utilized to determine the desired threshold limits for the design condition. Quantitative values shall be established for these limits that can be accurately translated into measurements that are easily and readily obtained in the field, which shall enable the designers and operators to evaluate the behavior and performance of the structure. A detailed discussion of the design assumptions shall be presented in the feature design memorandum. The threshold limits along with the predicted performance levels shall be addressed in the project instrumentation design memorandum and in detailed instructions to project personnel and any other personnel involved with the instrumentation.

6. HYDRAULIC OPERATION

The hydraulic operation of the project shall be performed as an analytical research type of work. This work shall generally cover the following activities:

- Forecasting river flow.
- Control of river levels.
- Best use of pondage.
- Functioning of hydraulic structures including desander flushing

6.1 Control of River

The headworks shall meet its obligations concerning the regulation of stream flow and maintaining downstream river condition similar to those before the construction of the project. Usually, the minimum natural flow to be released shall be based on the environmental guidelines agreed upon during project implementation stage. The upper water levels shall be limited to the acquired land area.

6.2 Spillway Operation

The spillway gates shall be operated according to a fixed schedule prepared in advance. This schedule shall be determined in part from design stage calculations and in part by observations in the field, considering the effect of various combinations of gate operations on tailwater levels and plant output. A warning system shall be installed and activated to inform the people downstream in case of sudden opening of the spillway gates.

In general, the gate operation shall ensure that:

- The required pond level is maintained both during the dry season flows and flood flows.
- The dry season flows remain near the undersluice bays so that the supply of water to the intake is not affected.
- As far as practicable, a fairly uniform distribution of discharge along the width of the spillway is obtained.
- Parallel flows both on the upstream and the downstream of the diversion structure are avoided.
- The risk of deep scours and shoal formations in the vicinity of the spillways, both on the upstream and the downstream is minimized.
- Maximum sediments/debris deposits on the upstream side are flushed downstream and entry of sediments in the intake is minimized.
- The hydraulic jump does not form beyond the toe of the downstream glacis in any case.
- A relatively high intensity of flow is avoided in the deep scour zones, if formed.
- Shoal formed on either upstream or downstream or both of the diversion structure are washed out and kept away from the structure.
- Constraints regarding rates of lowering/ raising of pond are considered in the gate operation schedule.
- Constant and regular supply to the river intake is maintained even during fluctuations.

In order to lessen downstream disturbance, the overflow from the spillway shall preferably be distributed in small amounts between alternate gates across the spillway section instead of allowing the entire overflow through one or two gates. In this connection, it shall be desirable to watch out for wide areas in the flood channel below the dam when sweeping eddies can form and augment the flow and scouring action in the main channel below the spillways. The contours of the banks subject to erosion shall be taken often enough to keep record of progressive changes due to spillway discharge.

6.3 Desander Flushing

In order to monitor the effectiveness of the desander, samples of water entering and leaving the desander shall be collected at least twice a day during the monsoon season in the initial years of operation. Results of the analysis of these samples shall be used to establish the desander flushing operations. Since the quantity and quality of sediments entering the turbine affect the operational life of the equipment, the flushing operation at the desander shall be carried out to ensure that there is no interruption in power generation.

7. OPERATION OF HYDRO-MECHANICAL INSTALLATIONS

All lift gates shall be operated at suitable intervals to free the mechanism and wash out extraneous material. During low flows when openings may not be desirable, raising of gates by 150 mm for a few minutes shall suffice. If the gates have not been moved for a sufficiently long time, they shall not be forcibly raised all at once but shall be lifted by about 30 mm or so and left at that position for about 10 to 15 minutes till the silt deposited against the gates gets softened and water begins to ooze out. This is essential to avoid heavy strain on the machinery.

In addition, the following procedures shall be adopted:

a. The speed of operation of the gates shall be limited to the maximum speed indicated by the manufacturer.
b. Intake gates shall be opened equally unless otherwise indicated by model studies due to adverse local conditions. gates shall be for optimum and structural
c. Barrage gates shall be subject to wedge operation, opening from the centre and moving on either side till all the gates have been opened equally. The gates shall be opened in installments not exceeding 30 mm at a time. Gate opening shall be suitably increased to allow passage of boulders.
d. Gate operations shall not jeopardize the safety of the structure at any time, and the permissible difference in static head on either side of the divide walls shall not be exceeded beyond the safe limit.
e. The gates operating silt excluding devices shall be closed very slowly to avoid water hammer action which can otherwise damage the structure.

8. INSPECTION AND MAINTENANCE

Inspection of headworks shall be performed to obviate the possibility of extension of damage. Any maintenance or repairs found necessary as a result of inspection shall be carried out well before the onset of the next monsoon.

The inspection and maintenance for different headworks structures shall be performed as indicated in the following sections.

8.1 Concrete Diversion Structures

Concrete diversion structures shall be regularly inspected for the following:

- evidence of piping, muddy water boils in the areas around dam monoliths;
- abnormal increase or decrease of flow from foundation drains, structural joints or face drains;
- any significant change in uplift pressures;

- significant cracking of mass concrete structures;
- excessive deflection, displacement or vibration;
- significant damage to any structure;
- spillway surface erosion by bed load movement or downstream damage;

The following activities shall be performed for the maintenance of concrete diversion structures:

- Drainage system in the foundation and the body of the diversion structure shall be maintained properly.
- Leaks, cracks and spellings on the surface of the diversion structure and in openings like gallery and adits shall be treated.
- Leaks, cracks, slides, etc, in the abutment shall be treated.
- Measures shall be taken to protect against harmful retrogression.

8.2 Embankments

Inspection of embankments shall aim at detecting the following conditions:

- sloughs, settlement or slides;
- evidence of piping, muddy water boils;
- increase in seepage quantities through or under embankments;
- any significant change in pore-water pressure either in embankments or their foundations or abutments;
- unusual vertical or horizontal movement or cracking of embankments or abutments;
- sinkholes or localized subsidence in the foundation of or adjacent to embankments.

Suitable preventive measures shall be adopted to address any of the above conditions.

8.3 Energy Dissipators

The surface of the energy dissipators shall be checked for cavitation and abrasion damage. Such damage shall be repaired in a timely manner to stop their propagation.

Any debris or rock pieces collected in the energy dissipation structures shall be removed before monsoon. Cleaning beyond these structures shall be done to the extent required.

8.4 Intake and Trash Racks

The intake shall be carefully examined every year in the dry season , and all necessary repairs shall be carried out in time. The trash racks at the river intake shall be kept clean of leaves, drifts of all kinds and ice at all times. Plants provided with mechanical or automatic racking systems shall pay careful attention on maintenance of these racking systems. Special grappling devices shall be provided in plants where heavy water logged timber, tree stumps and the like form the major portion of the debris stopped by the trash racks.

8.5 Desander and Sediment Excluders

The desander shall be emptied and inspected at regular intervals to check for damages caused by flushing sediments. A thorough inspection of roofs, ducts and mouth of the sediment excluders shall be carried out every year in the dry season. Minor repairs may be carried out under water and major repairs by local isolation.

8.6 Aprons

Sounding and probing of the apron shall be undertaken every year immediately after the monsoon in order to assess the scours and launching of aprons in the vicinity of structures.

The non-launching portion shall be carefully examined, particularly on the downstream side, to ensure the effectiveness of inverted filter.

8.7 Impervious Floors

A thorough inspection of upstream and downstream impervious floors shall be done after the monsoon. Careful inspection of joints shall be done for floors in boulder reaches. Minor repairs can be done under water whereas major repairs may be undertaken by isolating the area.

8.8 Gates

Gates that are not frequently in use, such as spillway gates, shall be tested before the onset of the monsoon season every year. Water leaks from gates shall be prevented. All cavities and angles in the gates/shutters shall be kept clear of debris, driftwood, moss and silt accumulations. All drainage holes in the webs of horizontal structural members shall be kept open and no water allowed to remain entrapped. Green stains shall not be allowed to form on the steel members at the back of the gates/shutters. The gates and counter balanced boxes shall hang perfectly level and plumb. This shall be checked occasionally and adjustment made as needed. In case of shutters, the chains/anchors holding them shall be kept free from rust.

8.9 Gate Grooves

Grooves, and particularly their machined faces, shall be kept clean and lubricated well and all sticky deposits shall be scraped off before application of lubricant.

8.10 Seals

Seals of gates shall be checked from time to time. Efficiency of rubber seals shall be tested initially after construction and at the time of closures or isolation of different portions for repairs. The horizontality and verticality of the seal seat and wall plates shall be checked with spirit level and seal faces of the rubber seal shall be tested to press uniformly both by light test and by use of paper strip inserts. Seals of the gate shall be checked for wear and tear and deterioration. These shall be adjusted/replaced as necessary.

8.11 Steel Wire Ropes

All steel wire ropes must be cleaned to remove all dust accumulation and lubricated with suitable greases at least once a year. The portion of steel wire rope which is submerged in water shall be lubricated frequently, preferably thrice a year.

8.12 Roller Trains and Fixed Rollers

The roller trains shall be examined at least once a year. Partially jammed rollers shall be cleaned, freed and greased but totally jammed rollers shall be replaced. The bolts of roller guard shall be checked and tightened. The sliding/fixed rollers shall be extracted at the time of closure (unless necessitated otherwise due to some defects which may need immediate repairs), and cleaned and greased properly. Worn out pins shall be replaced and suitably held against rotation by filling the empty space between the pin and the side plates through welding or by other approved means. Spare rollers shall be kept in stores for ready replacement.

8.13 Winches/Hoist

All winches and lifting drums shall be examined at least once a year to see if all the gears and axles are clean and properly lubricated. All grease-fed bearings shall be cleaned, old grease removed with kerosene oil and fresh grease applied. The alignment of shafts shall be checked and coupling bolts tightened.

9. INSTRUMENTATION AND MONITORING

Monitoring of instrumentation and other performance indicators shall be performed regularly.

9.1 Uplift Pressure

Uplift pressure shall be measured and compared with the design uplift pressures, and needed remedial measures shall be adopted. The frequency of observation shall depend upon local conditions. It may generally be enough to take observations once a month during monsoon period and more frequently during the non-monsoon period.

9.2 Pressure Release (Drainage) Pipes

To check their working, pressure release (drainage) pipes, where provided, in the downstream floor shall simultaneously be checked for the quantity and quality (sediment content) of the discharge. Such observations may be possible only during the dry season when all the gates of the compartments are closed.

A correlation between head of water and discharge shall be established, and any large variations shall be immediately taken care of. As discharge of sediment in the effluent could lead to undermining of the foundations, immediate remedial measures shall be undertaken. In extreme cases, it may become necessary even to completely block the sediment discharging pipe.

9.3 Hydraulic Jump Profile

Strip gauges shall be painted every 10 m on the wing walls and the long divide walls to observe the hydraulic jump profile in the prototype under different hydraulic conditions. The following observations shall be taken:

1. Upstream water level
2. Downstream water level
3. Shade temperature - maximum and minimum
4. Temperature of the river water at a depth from the surface below which it remains approximately constant
5. Temperature of the sub-soil water in a few selected observation pipes
6. Water level in all pipes may be observed by using a bell sounder or by other suitable means
7. Discharge from drainage pipes
8. Depth of sediment on upstream and downstream floors.

9.4 Suspended Sediment

During the monsoon season, water sample shall be taken simultaneously upstream and downstream of the undersluices and in the desander to assess the suspended sediment therein. Such observations shall be taken at least once a week (closer intervals in case of high sediment concentration) to judge the efficiency of sediment exclusion and to decide if any change in the mode of regulation and/or other remedial measures are required.

9.5 Settlement

Where appreciable foundation settlements are anticipated, particularly when the structure is founded partially or wholly on clay or other soft soil, surface settlement of relatively heavily loaded parts of the structure shall be observed early in the cold weather every year and remedial measures undertaken, if necessary. This can be done by establishing permanent observation points of steel on the structure and doing precise levelling from permanent bench marks established sufficiently away from the influence of any structure.

9.6 Retrogression

Retrogression of the river bed can be expected downstream of the diversion structure. In order that the lowering of water level at any discharge condition does not exceed that provided for in the design, it is necessary to establish gauges on both banks, one immediately downstream of the work and two more, 1000 and 2000 m downstream of the first and to observe them simultaneously at least once a day. Remedial measures shall be undertaken as and when required to ensure the safety of the structure

9.7 Aggradation Upstream

The river bed upstream of the barrage of diversion structure is likely to aggradate resulting in increased afflux and reduction in freeboard provided in design. To determine the increase in the afflux, if any, gauges shall be established on the upstream, one immediately upstream of the work and one each at 1,000 and 2,000 m upstream of the first and observed regularly. Afflux bunds may have to be raised, if found necessary, to restore the designed freeboard.

9.8 Discharge Distribution and Crossflow

Observations shall be taken to find the discharge distribution through different bays of the overflow section. If there is significant crossflow and/or difference in discharge intensities through different bays, remedial measures shall be taken to check this tendency for which improved regulations may be of great help.

9.9 Pond Capacity

For pondage run-of-river projects, soundings in the pond area may be made at suitable intervals for periodic review of storage capacity.

10. DATA COLLECTION

An integral and important of operation and maintenance shall be the collection of hydrological, meteorological and silt data at the project. Collection and analysis of such data shall also be desirable for future design of headworks.

10.1 Hydrological and Meteorological Data

The operation plan of a power plant shall provide for an adequate system of forecasting river flow and power available. This shall require the establishment of rain gauges and river gauging stations. The number and location of the rainfall and river gauging station shall depend on the size of the watershed and type of the plant.

Provision shall be made for installation of automatic gauges for water levels about the plant, particularly the headwater and tailwater levels. These gauges shall be installed at locations least influenced by the velocity of flow to or from the power plant. Readings from these gauges shall be utilized for preparation of more accurate rating curve to help in better operation of power plant.

Where the pool is large and computation of inflow involves estimating daily pondage, it is advisable to install a wind-velocity and direction recorder so that the natural pool level without wind may be determined with fair degree of accuracy.

Therefore, each power plant shall establish and operate at least one hydro-meteorological station, consisting of rain gauge, humidity, wind velocity and direction meter, sunshine meter, temperature and evaporation pan. At least one or preferably two automatic gauging recorder shall be established in upstream unaffected reach of river downstream after tailrace water flow in the river is normalized.

10.2 Silt Data

Silt sampling in the river during the monsoon season shall be carried out to collect the following information:

- Concentration of suspended sediments in the water flow
- Particle size distribution of suspended sediments
- Bed-load transport rates
- Particle size distribution of the bed-load
- Particle size distribution of the riverbed material/armoured layer
- Mineralogical and petrographical composition of the sediment load, in practical terms the contents of hard minerals like quartz, garnet and felspar
- Content of organic matter in the suspended load
- Density of deposited sediments.

Analysis of the quantity of silt in the river shall be used for a correct assessment of reservoir silting. For a run-of-the-project, regular silt sampling at intake and outlet of desander shall be required to assess the effectiveness of the desander and estimate the damage that could occur to turbines.

11. INFORMATION DATABASE

DoED shall establish and maintain a country-wide database which shall include data and information on projects under construction or operation. This database shall be utilized by DoED to build up its own technical expertise and administrative capabilities. It shall be openly and easily accessible to all so that it can serve as valuable information for concerned organizations and future projects in developing rational design, construction and operation bases suitable to Nepali conditions.

As a minimum, the database shall include the following information:

- periodic inspection schedules and history of project remedial measures including project deficiencies, status of deficiencies, completion dates, estimates, actual expenditures;
- data and information gather from the monitoring, evaluation and inspection of different projects under construction or operation;
- problems encountered in different projects and their remedial measures, and
- operational mishaps and accidents at projects.

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