# CHAPTER - 3

## BEST PRACTICES IN PLANNING, DESIGNS & CONSTRUCTION OF CIVIL WORKS OF HYDRO POWER PROJECTS

> ❝*This chapter gives a detailed step by step procedural account of various stages involved in planning, design and construction of civil works of hydro power projects. Topics like types of hydro power projects, components, power house installation, types of dams and the related geo-mechanics, instrumentation, measurements of features like gates, barrages and tunnels, safety assurances, maintenance and remedial measures for ensuring optimum performance levels have been dealt in quite detail. Each step involved in setting up of a hydro power project is elaborated with appropriate references and precise mathematical formulae.*❞

### 3.1 INTRODUCTION

**3.1.1** Hydro Power Projects harness the potential energy of the water at a relatively higher elevation, which is converted first into mechanical energy and then into electrical energy. This transformation of energy from one form to another is facilitated by the water in motion. This journey from



*120 MW Tanakpur Power Station (Uttaranchal) - Barrage*

water to watts is through water conductor system, which may comprise of intake structure, trash rack structure, desilting chamber, head race channel / tunnel, surge chamber / shaft, penstock, power house (spiral case, turbine, draft tube etc.) & finally tail race channel / tunnel. This chapter deals with the basic design philosophies & best practices in vogue in planning, design & construction of these civil works, which facilitate the journey from water to watts.

**3.1.2** Hydel civil design is unique and depends primarily on site specific features, viz., topology, geology, hydro-meteorology etc. No practice can, therefore, be generalized in case of hydel civil design and presented as best practice. As such, one has to begin with basic classification system and their general characteristics to evolve best practice suitable for a specific site condition.

## 3.2 STAGES OF IMPLEMENTATION

Best Practices in planning, design and construction of Civil works in Hydro-Power Projects are taken care of at the following four stages:

- Feasibility report stage
- Detailed project report stage
- Tender preparation stage
- Construction stage design and implementation stage

These four stages are described as below:

### 3.2.1 Feasibility report stage

During basic planning of the project besides engineering issues, environment, ecology and other socio – economic factors also play important role. However here only the conceptual and engineering issues are detailed:

i)      Detailed study of hydrology, which includes water availability studies, flood studies and collection of historical hydrological data and meteorological data.

ii)     The installed capacity of the project along with design head and the design discharge for the project.

iii)    Fixation of location of the dam and the powerhouse.

iv)     Fixing of the height of the dam above riverbed.
v)      Fixing the preliminary layout of the water conductor system.
vi)     Fixing the type of the power house
vii)    Working out the design energy for the project.
viii)   Evaluation of infrastructure works likely to be involved.
ix)     To work out the knowledge of civil design, electrical design, hydrology, geology, construction and cost engineering is very important to arrive at a good feasibility report.

## 3.2.2  Detailed Project Report stage

Detailed project report is the outcome of detailed investigations and studies carried out at the site.

These detailed investigations essentially include the detailed information on subsurface geology along with detailed hydrological studies based upon the historical hydrological data available.

The following essential tasks are performed during the DPR stage design

i)      Precise study of hydrology.
ii)     Detailed investigation and study of geology.
iii)    Decision on the type of dam
iv)     Fixation of spillway design flood
v)      Fixation of spillway system and the sizing of the same.
vi)     River diversion scheme.
vii)    Fixation of the layout of water conductor system.
viii)   Fixation of size of HRT.
ix)     Fixation of the type of intake and desilting arrangement.
x)      To fix the location of the surge shaft based upon topographical and geological information.
xi)     To fix the size of the surge shaft, based upon transient studies.
xii)    Fixing the detailed layout of powerhouse complex including location of access tunnels and ventilation tunnels.
xiii)   To work out powerhouse planning in consultation with E&M group.
xiv)    To work out the details of TRT outlet and its joining with the river
xv)     To work out details of HM equipments proposed for the projects.
xvi)    Detailed assessment of infrastructure and construction machinery requirement.

xvii) Assessment of quantity of construction material, based on the volume of concrete involved in the project and identification of potential quarry sites.

All these tasks are integrated in nature and therefore a multidisciplinary interactive approach is required to prepare the scheme

Topographical and geological information and river morphological character is specifically taken into account, while selecting a particular type of structure for the project. The layout of the project is considered in totality.

Besides fixing the sizes and layout of the components, the construction processes i.e. execution of the structures, support during excavation and concrete placement aspects are taken into consideration.

The good practices involved in the planning and design of hydroelectric project at DPR stage therefore include complete integration of inter-disciplinary knowledge. An organization's past experience, adequate staffing in terms of specialized engineering groups and exposure to new construction and design practices are essential inputs for a good design.

### 3.2.3 Tender preparation stage

Each project is divided into a number of tender packages for execution. It is common to have two or three packages for civil works and at least one package for HM works. The tender preparation stage for each package includes the following:

i) Reviewing and firming up the work done at DPR stage
ii) Preparation of detailed drawings for tender.
iii) Preparation of technical specifications
iv) Preparation of Bill of quantities
v) Preparation of commercial conditions and bid proposal sheets

During the tender stage, the works, which are likely to pose challenges during construction, are particularly identified. An appropriate strategy to meet these challenges on the basis of past experience needs to be evolved.

Since time and cost overrun along with geological surprises/difficulties

are considered to be the major concerns in the execution of hydro projects, there is a need to identify the challenging tasks and insisting on pre-qualification to identify technically competent agencies for them. Expected variations to take care of these challenging tasks should be reflected in the Bill of Quantities.

Some important items requiring attention in the tender preparation stage are given below:

i)      Excavation below river bed for the dam
ii)     Foundation sealing of coffer dams
iii)    Excavation of surface slopes for surface structures and their support.
iv)     Excavation and support of tunnels particularly in poor class of rock. Ground water table and its likely effect.
v)      Excavation of large diameter shafts, such as surge shafts particularly when geological conditions are not favorable
vi)     Excavation of deep vertical/inclined shafts.
vii)    Excavation and support of large under ground openings such as powerhouse cavern and desilting basin cavern.
viii)   Preparing of the construction schedule in consultation with the specialized agency for particularly identified difficult tasks based upon the known geological information the construction planning, which includes selection of specialized agencies, requirement of specialized equipments & materials as well as construction methodology,
ix)     Sufficient provision of investigation during construction especially for underground works

Some of the items that require attention and should be provided based upon past experience are given as under:

i)      Dewatering of surface and underground works during construction.
ii)     Provision of quantities for grouting works both for dams and tunnels.
iii)    Location of muck disposal sites vis-à-vis the point of generation of muck.
iv)     Location of quarry materials from the point of its functional efficiency.

Tender preparation stage should be more construction oriented and should be made in great details.

### 3.2.4 Construction Stage (implementation stage)

Besides performing structural calculations, design and detailed dimensioning, the construction stage design includes adaptation of the design corresponding to the situations arising during the construction process. Therefore, regular interaction between the design groups, the site team and the contractor at site is a must to adapt to the construction. Data or information generated during construction should be fully shared among contractor, site team & the designer and should be documented for future use.

### 3.3 TYPES OF HYDRO POWER PROJECTS

**3.3.1** Depending upon the characteristics, the river hydroelectric projects are possible with different layouts and for different installed capacities. The two distinct layouts, which are used are:

a)     Layout with a power house in the vicinity of the dam i.e. P. H. with a short water conductor system.

b)      Layout with a dam, a long water conductor system and the power house. This configuration invariably involves either a head race tunnel alongwith a surge shaft and pressure shafts/penstocks or a power channel with forebay and penstocks. Power house is located much away from the dam.

The powerhouse can be a surface powerhouse or an underground powerhouse, in both these arrangements.

**3.3.2** Hydro power projects can be broadly classified into three categories from their placement point of view;

(a)     **Surface Power House** – All components of the hydro power projects are on the natural / excavated ground surface. This has the advantage of pre-determined topographical design and is easy to construct. However, this has the disadvantage of limitation of head available as per the topography.

(b)  **Underground Power House –** All major components of the hydro power projects are underground in tunnels & cavern. This is very advantageous as in favourable geological settings it overcomes the limitation of head available as per the topography and provide compact & economical layout. However, this has the disadvantages of geological uncertainties resulting in indeterminate design & construction problems leading to time & cost over runs.

(c)  **Semi-underground Power House** – Some components of the hydro power projects are underground, while others are on surface. Sometimes the advantages of both surface & underground power houses are clubbed together in a semi-underground power house, provided topography & geology so permit.

**3.3.3** Hydro power projects can also be broadly classified into three categories from their operation point of view;

a)  **Run-of-river project –** As the name implies, the project is planned as run of the river. Water is diverted from the river, routed through the project & finally ploughed back to the river at a lower level downstream. It takes advantage of the drop in elevation that occurs over a distance in the river and doesn't involve water storage. Conceptually very simple, but it has one big problem, i.e., power generation fluctuates with the river flow and the firm power is considerably low, as it depends on the minimum mean discharge. Canal power projects are also run-of-river projects.
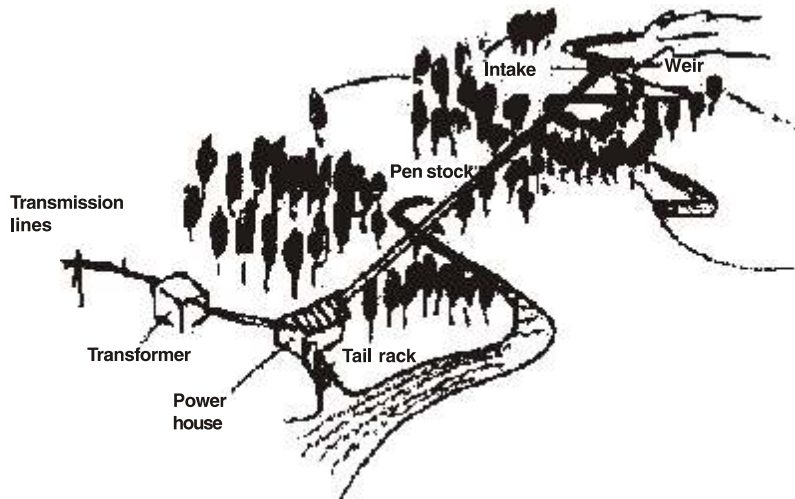
**(b)** **Storage Project** - Storage project provides storage or pondage, and thereby, evens out stream flow fluctuations and enhances the water head. Thus, it increases firm power & total power generation, by regulating the flow. However, providing storage is complicated & costly affair as it involves construction of dam as well as rehabilitation to hydro power projects.
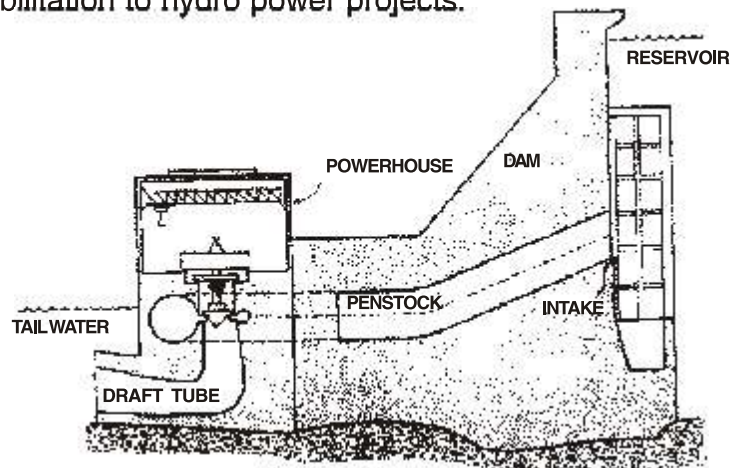


**(c)** **Pump - Storage Project -** Pump - Storage projects store the surplus energy in the power grid system during the off-peak hours and utilize the same energy during the peak hours. These involve reversible turbines, which can generate power from water of upper reservoir during peak hours & pump back water from lower reservoir to the upper reservoir during off-peak hours. These are advantageous in power grid systems of mix type, which have thermal & nuclear power houses in addition to hydro power projects.
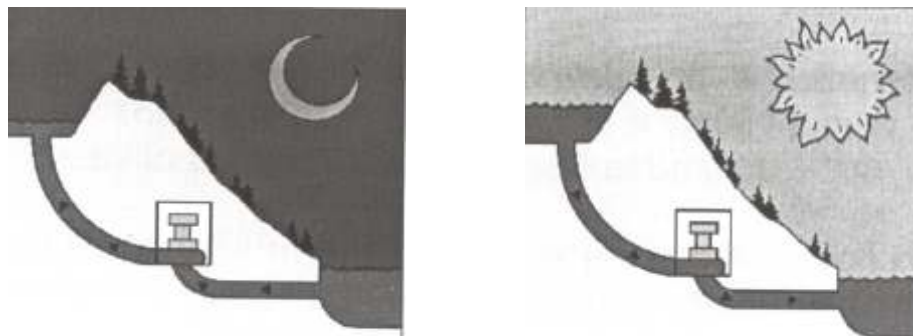
**3.3.4** Selection of a particular type of Hydro power project depends primarily on techno-economic considerations, which varies from case to case basis. This is carried out only by undertaking a detailed study comprising of following:

> Topography and geomorphology of the site
> Evaluation of the water resource and its generating potential
> Site selection and basic layout
> Hydraulic turbines and generators and their control
> Environmental impact assessment and mitigation measures
> Economic evaluation of the project and financing potential
> Institutional framework and administrative procedures to attain the authorizations.
> Magnitude of Rehabilitation & Resettlement (R&R)

## 3.4 COMPONENTS OF HYDRO POWER PROJECTS

The journey of water to watts is performed through various components, which in turn depend on various techno-economic considerations that vary from case to case basis. It is primarily the type & layout of the components, which leads uniqueness to each project. Three basic elements are necessary in order to generate power from water: a means of creating head, a conduit to convey water and a power plant. To provide these functions, the following common hydro civil components are used:

**3.4.1** **Dam / Barrage, Spillway and River diversion-** Depending upon the layout, the diversion structure can be a barrage or a dam to create the head. The spillways are used for discharging surplus water as per requirement.

**3.4.2** **Intake Structure** - A structure to divert water to a waterway, which includes **trashracks**, a **gate** and an entrance to a canal / tunnel, penstock or directly to the turbine depending on the type of project.

**3.4.3** **Head Race Channel -** A canal, tunnel and/or **penstock**, which carries the water to the powerhouse. Sometimes a **desilting chamber** precedes the Head Race Channel, which removes the larger sized sediments into the turbines.

**3.4.4** **Surge Tank -** A surge tank to absorb water hammer effects due to rapid start or closure of the turbine.

**3.4.5** **Power House -** A power house structure, housing **turbines**, **generators** etc. It also houses the entrance (**spiral case**) and exit (**draft tube**) of the turbine, which include the valves and gates necessary to shut off flow to the turbine for shut-down and maintenance.

**3.4.6** **Tail Race Channel** - A tail race canal or tunnel, which carries the water from the turbine exit (draft tube) back to the river.

**3.5** **BEST PRACTICES FOR INDIVIDUAL COMPONENTS**

**3.5.1** **Dams, Spillways and river diversion.**

Depending upon the layout, the diversion structure can be a barrage, a diversion dam or a high dam (height above foundation level of the order of 100-200 m). A barrage (a gated structure across river founded in alluvium) is normally considered, when it is not possible (due to topographical or other constraints) to provide much pondage and water level is maintained at constant, for diversion of flow. Diversion dams, besides diverting flow in the water conductor system, also provide pondage for diurnal, storage of water in the reservoir to meet peaking requirement of generating station. Depending upon the characteristics of the site, geology and other factors, the dam can be either a concrete dam, an embankment dam or combination of both.

When concrete dam is adopted, spillway is invariably incorporated in the centre of dam. In the rockfill dam layout, for narrow valley, the spillway is accommodated on one of the banks. Size of the spillway depends upon the design flood to be passed, which depends upon the catchment area and amount of precipitation in the river basin.

Depending upon the type of dam and design flood to be handled during construction, different diversion schemes with diversion tunnel/channel and upstream and downstream cofferdams are adopted.

Silting of reservoir is a major problem in hydroelectric projects. To take care of the problem, in modern layout, the low level spillways equipped with radial gates are provided. This allows the power intake to be located sufficiently above the spillway crest. Some of the example projects with low level spillways are Chamera-1, Dulhasti, Nathapa Jhakri, Dhauliganga, Teesta-V. Annual flushing of reservoirs is possible only with

*Chamera Hydro Power Station, Stage-I, 540 M (HP) Dam*

low level spillways.

For high dams when large design flood is to be passed, two tier spillways are considered. With rockfill dam layout, the low level spillways are sometimes provided in the form of tunnel spillways. Final design of spillways and the energy dissipation system is evolved after carrying out hydraulic model studies.

### 3.5.1.1 CONCRETE & MASONRY DAMS

Gravity dams resist water thrust and other overturning forces by their self weight alone. Both concrete and masonry gravity dams have been constructed in India . As the flood discharges in our country are generally quite high, large spillways are required for passing the flood discharges. Common types of spillways envisaged are ogee spillways or chute spillways. They form a part of both embankment as well as gravity dams. Ogee spillways & the crest structure of chute spillways are designed as gravity dam overflow sections.

Masonry dams had been quite popular in our country essentially because

of the following reasons:

- Savings in cement as the mortar constitutes only about 45% of the total volume of masonry
- No temperature control measures are required which are necessary in concrete dams
- They are less prone to Alkali-Aggregate reaction
- They are labour intensive with high employment potential

Earlier lime-mortar was being used in our masonry dams, as cement was scarce.  After the availability of cement improved, it has virtually replaced lime.  However, on account of poor workmanship and non-availability of good masons the quality of masonry dams is going down these days.  In many of our masonry dams there are seepage problems. Earlier a 3.0 m thick random rubble masonry in rich cement mortar (1:3) with coursed chisel dressed facing was normally provided on the u/s face of masonry dams.  In view of the seepage problems now faced in many masonry dams, the following measures are being taken / considered for controlling seepage through them:-

- Guniting on u/s face
- Face Concrete Membrane on u/s face
- Sandwiched Concrete Membrane on u/s face
- Prepacked Sandwich Concrete Membrane on u/s face

These arrangements are discussed in detail in IS: 11155.  There is also a view, which is gaining ground that we should increasingly go in for concrete dams instead of masonry dams in view of the seepage problems because of poor workmanship and the difficulties in controlling the same.

Design issues considered in the planning, design and construction of concrete dams are:

- Incorporation of spillways
- Earthquake resistant design.
- Mix design for the mass concrete.
- Evaluation of foundation conditions and their foundation treatment.
- Concrete production and placement system.

Traditional Concrete Gravity dams envisage block-wise construction.  The

placement of concrete is by Tower Cranes, Cableways etc. Large mixing/ batching plants are deployed with measures for temperature control of concrete by way of mixing ice, use of refrigerated water, cooling the coarse aggregates etc. In our country the pre-cooling method is popular. The lift height, time interval between successive lifts, placement temperatures etc. are worked out as per IS 14591-1999 - Guidelines for Temperature Control of Mass Concrete for Dams. Longitudinal contraction joints are avoided. The maximum aggregate size used in the hearting is generally 150 mm. Low Heat Cement, Ordinary Portland Cement (OPC) + flyash, Portland Pozzolana Cement (PPC), Portland Slag Cement (PSC) are invariably considered for controlling the heat of hydration. The BIS codes now permit

- Flyash upto 35 %  in PPC
- Slag    upto  65 % in PSC

Most of our gravity dams have plane ungrouted transverse contraction joints without keyways. Each block is designed as a separate monolith. The stability analysis is carried out as per IS – 6512 for various load combinations. For seismic conditions the pseudostatic analysis is done as per IS 1893. For final designs especially for dams located in seismically active areas our code prescribes a Dynamic analysis. There are many softwares available for the same. Some of the commonly used softwares are EAGD 84, NISA, NASTRAN, ABACUS etc. Seismic input for these studies viz. response spectrum, accelerogram for different levels of earthquake motions eg. MCE, DBE, OBE etc. are worked out by institutions like CWPRS, Department of Earthquake Engineering, University of Roorkee etc. Sometimes simplified dynamic analysis is carried out as per the technical paper "Dynamic method for Earthquake Resistant Design & Safety Evaluation of Concrete Gravity Dams" by Anil K. Chopra & Charles F. Corm published in ICOLD, New Delhi 1979 which considers the fundamental mode of vibration. This has subsequently been refined to include the higher modes of vibration , effect of foundation-rock interaction and the effects of reservoir bottom materials (Fenves and Chopra, 1986) in addition to the effects of dam-water interaction & water compressibility considered earlier. The refined simplified analysis procedure is published in the report No. VCB/EERC-85/10, University of Berkeley, USA.

The basic foundation treatment envisages provision of curtain grouting, consolidation grouting, drainage arrangements etc. This is provided based on various BIS codes and prevailing practices taking due cognizance of actual site-specific geology. Many of our dams are required to be constructed on foundations with adverse geological features like shear zones, faults etc. and at sites with possibility of sliding stability along horizontal and d/s dipping shear seams/zones etc. Site-specific foundation treatment is required to be worked out. Specialist literature like ICOLD publications, Technical papers, Technical Monographs, publications of reputed world-wide institutions like USBR, U.S. Army Corps of Engineers, Tennesse Valley Authority etc. are referred to for this purpose .

The shaping of foundation profile is generally carried out as per IS:11155 – "Code of  Practice for Construction of Spillways & Similar Overflow Structures".

However, the latest trend in the world is to go in for Roller Compacted Concrete (RCC) dams instead of Conventional Concrete dams, as they are more economical and can be constructed very rapidly. The construction of a RCC dam is done from abutment to abutment using Embankment dam construction equipments/techniques. Relatively lean concrete of no-slump consistency is spread horizontally by bulldozers in layers not exceeding 300 – 600 mm in thickness and compacted by vibratory rollers.

RCC dams can be broadly classified as under:

i)      Lean RCC dams (Cementitious Content < 99 kg/m³)
ii)     Medium paste RCC dams (Cementitious Content = 100 to 149 kg/m³)
iii)    High paste RCC dams (Cementitious Content  < 150 kg/m³)
iv)    RCD dams (As constructed in Japan)
v)     Hardfill dams

The proportion of various types of RCC dams completed/under construction and the cementitious content used by the leading RCC dam countries are given in tables 1 & 2 below:

Table 1 : Percentages of various types of RCC dams built/under construction
(Data up to 1998 ) - Dunstan

| Category | % |
|---|---|
| High paste content RCC dams | 46.5 |
| Medium paste RCC dams | 21.5 |
| RCD dams | 18.0 |
| Lean RCC dams | 12.5 |
| Hardfill dams | 1.5 |

Table 2  :  Cementitious content in RCC dams in the leading RCC dam countries
of the world (Data up to 1998 ) - Dunstan

| Country | Number | Cement Content $(kg/m^3)$ | | Pozzolana Content $(kg/m^3)$ | | Total cementitious content $(kg/m^3)$ | |
|---|---|---|---|---|---|---|---|
| | | Avg. | Max. | Avg. | Max. | Avg. | Max. |
| China | 36 | 79 | 140 | 94 | 140 | 173 | 230 |
| Japan | 36 | 87 | 96 | 35 | 78 | 123 | 130 |
| USA | 29 | 85 | 184 | 53 | 173 | 138 | 252 |
| Spain | 21 | 75 | 88 | 130 | 170 | 204 | 250 |

It may be seen from the above statistics that the present trend is to go in for High paste content R.C.C. dams.  These dams also permit use/ utilization of fly ash in a very big way. The fly ash is required to conform to IS: 3812 .Earlier the practice was not to provide any transverse contraction joints in R.C.C. dams.  But after the experience in Upper Stillwater (USA) in which cracks developed in the dam, the present practice is to provide transverse contraction joints.  All RCC dams in Japan have joints at 15 m c/c.  The spacing of joints in RCC dams has ranged from 10 to 75 m (in those dams that have joints).

The following three methods have been utilized for forming contraction joints in RCC dams :

i)      Post-forming the contraction joints by vibrating steel or plastic crack intruders into RCC after spreading RCC (in RCD dams and in some RCC dams) or after compaction (most RCC dams).  This method has been used in 70% of the RCC dams.

ii)   Formed contraction joints against form work in a similar fashion to traditional concrete dams.  This has been used in 15% of RCC dams.

iii)  Using various methods for incorporating a plastic sheet in RCC during spreading, occasionally by placing the sheet over the steel frame. This has been used in 10% of the RCC dams.

Various practices are in vogue with regard to forming of faces of RCC dams, bonding of successive lifts, cold joint treatment, maturity factor concept etc.  Details are available in the recent ICOLD bulletin on "State-of-art of RCC dams".

In India  RCC dams are being built for the first time in Ghatghar Pumped Storage Project (Maharashtra) with following features :

● Lower dam (84m high)
● Upper dam (14.50m high)
● Saddle dam

Alkali-Aggregate Reaction had been a cause of great concern in concrete dams.  It is a chemical reaction between the active silica component in certain types of reactive aggregates and the alkalis mostly derived from cement giving rise to the formation of expansive alkali-silica gel which results in expansion of concrete, cracking etc. in the dam.

Hirakud & Rihand are examples of concrete dams suffering from this problem in our country.

The solution lies in the use of low alkali cement (Alkali's content < 0.6%), use of pozzolana/flyash and use of non-reactive aggregates for construction.  Petrographic examination of aggregates and testing of aggregates as per ASTM Standards is carried out in various reputed laboratories/institutions for establishing the non-reactivity of aggregates before use in concrete dams.

It is well known that most of our hydraulic structures are prone to damages on account of abrasion and cavitation.

The types of finish required on surfaces on which water flow is there, is prescribed generally as per  Concrete Manual of USBR.

Cavitation Index is the parameter, which is used to predict the possibility of cavitation.  IS-12804 – Criteria for estimation of Aeration demand for Spillways and Outlet Structures (presently under revision) is often used for the purpose.  Besides this, other specialized literature like USBR

Engineering Monographs No. 41 & 42 are also referred to. Hydraulic model studies are carried out to design the aerators for spillways. To safeguard against cavitation damages, aerators are being planned/provided in many of our spillways. Three aerators are being provided in Tehri dam chute spillway based on extensive model studies.

In many dams, the spillway surfaces/glacis, energy dissipation arrangements are prone to abrasion erosion on account of the suspended silt load, rolling boulders etc. In some of our spillways steel rails at close spacing have been provided to provide safety to the concrete spillway surface against rolling boulders eg. in Chukha project. After the experience in some of our high dams in which the concrete surfaces have got damaged, it is being increasingly felt that we should go for high strength concrete like silica fume concrete (using super plasticisers) with compressive strength of the order of 50 to 60 MPa on spillway glacis/EDA of our very high dams with the idea of reducing maintenance problems later which can be quite expensive & difficult. High strength concrete is being proposed for Tehri spillways.

As regards Energy Dissipation arrangements the practice is to go for :

- Stilling basins
- Bucket type energy dissipators
  - Solid roller bucket
  - Slotted roller bucket
  - Flip Bucket

After damages to stilling basins at Libby & Dwarshek dams in the USA, presumably due to high hydrodynamic pressures, a lot of research has been carried out in that direction all over the world. Sensors have been used to measure these pressures in hydraulic models for some of our important dams like Sardar Sarovar, Tehri etc. Our BIS Code on Structural Design of Energy Dissipators ( IS-11527 ) is also under revision. Method for estimating hydrodynamic pressures is being included. Specialist literature in this regard needs to be consulted for evolving a safe design.

As regards bucket type of energy dissipators the tendency is not to go in for slotted roller bucket as its teeth are vulnerable to damages due to uneven flows and other reasons. Bargi, Jawahar Sagar etc. are some examples in which the teeth of the slotted roller buckets have got damaged.

For flip buckets the present practice is to provide a plunge pool at least in cases of high dams. Various formulae are there for estimating the depth of scour. Our IS Code on Hydraulic design of energy dissipators (IS-7365) and other specialized literature are referred to for the purpose.

## 4.5.1.2 ROCKFILL DAMS

Sometimes, characteristics of site are more favorable for construction of a rockfill dam instead of a concrete dam. However in such a case, spillway has to be planned separately.

Separate construction of spillway (specially low level spillway) may require surface excavation in rock, when valley is narrow, which results in high cut slope. Excavation, rock support and slope in overburden require considerable skill and mobilization.

In the planning of rockfill dams, material out of compulsory excavation are used to the maximum extent. River handling requirements of rockfill dam are more demanding as rockfill dams are required to be protected from monsoon floods during construction.

Concrete face rockfill dams are particularly preferred these days instead of earth core rockfill dams as these are intrinsically safe compared to earth core dams and allow greater flexibility and speed in construction and less affected by rainy season.

## 3.5.2 Intake Structure

A water intake must be able to divert the required amount of water into the power canal / tunnel or into the penstock without producing a negative impact on the local environment and with the minimum possible head loss. The intake serves as a transition between a stream that can vary from a trickle to a raging torrent and a controlled flow of water both in quality and quantity. Its design, based on geological, hydraulic, structural and economic considerations, requires special care to avoid unnecessary maintenance and operational problems that cannot be easily remedied and would have to be tolerated for the life of the project. Thus, the functions of an Intake Structure are;

(i)     Prevent entry of trash, debris, ice, boulders, logs of wood etc. into the conveyance system. This is achieved by providing a trash rack at the entrance.

ii)     Control the flow of water into the conveyance system by providing a gate or a valve.

iii)    Enable smooth, easy and turbulence free entry of water into the water conductor system. This is achieved by providing a bell mouth entry at the inlet mouth. This also enables to minimize the head loss at entrance.

iv)     Minimise sediment entry from the river in to the water conveyance system. For this purpose, special devices like silt traps and silt excluders are provided.

A water intake designer should take three criteria into consideration:

➢   **Hydraulic and structural criteria** common to all kinds of intakes;

➢   **Operational criteria**, e.g. percentage of diverted flow, trash handling, sediment exclusion, etc- that vary from intake to intake

➢   **Environmental criteria**, such as fish diversion systems, fish passes - characteristics of each project.

## 3.5.2.1 Water intake types

The first requisite is to decide about the intake needed for the given schemes. Different types of intakes available for consideration are as follows :

➢   The intake supplies water directly to the turbine via a penstock. This is what is known as **power intake** or **forebay**.

➢   The intake supplies water to other waterways, i.e., power canal, flume, tunnel, etc. that usually end in a power intake. This is known as a **conveyance intake**.
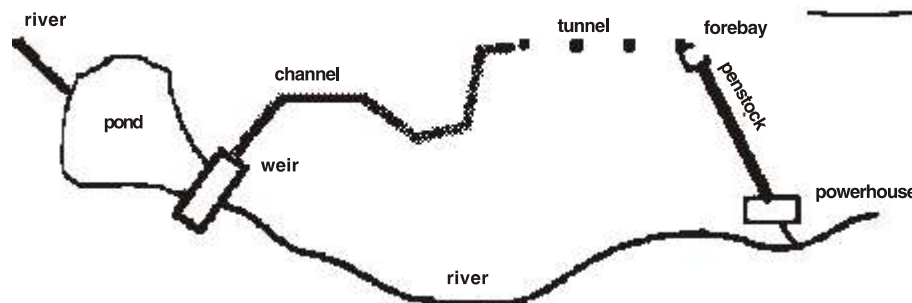
➢ The scheme doesn't have any conventional intake, but makes use of other devices, like **siphon intakes** or **trench** or **drop intakes**.



**Beginning of the open canal**
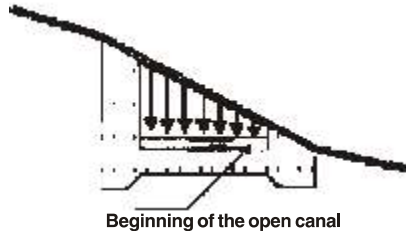
These are also classified depending upon the type of power plant and its layout, as under:

**Run-of-river intakes** – These intakes are generally conveyance intakes, provided in run-of-river plants. In small hydro projects, run-of-river type intakes consist of special devices, such as siphon intakes or trench or drop intakes.

**Canal Intakes** - These intakes may be of power or conveyance type intakes, provided in canal hydel projects.

**Reservoir Intakes -** These intakes may be of power or conveyance type intakes, provided in reservoir hydel projects. Depending upon the head above the center line of penstock, the reservoir intakes are categorized as;

a. **Dam Intake** – This is power or forebay type intake, provided in the dam body itself. Concrete & masonry dams can have dam intake.

b. **Intake in reservoir independent of dam** - This is conveyance type intake, provided in reservoir independent of dam body. Earth & rockfill dams can't have intake structures inside their body, so these are generally provided with independent intake.

c. **Re-entrant Intake** – This type of intake is adopted
   ● On upstream face of dam
   ● In open channel with flat bottom; and
   ● Where, the width of dam is inadequate to accommodate the intake

d. **Shaft or glory hole Intake** – This type of intake comprises of vertical shaft constructed in the reservoir, which carries water to the pressure tunnel.

e. **Tower Intakes –** A tower type structure is needed to house intake structure, when intakes can't be provided on the upstream face of the dam. Hence this is called tower intakes.

### 3.5.2.2 Location & Orientation of Intake

The location of the intake depends on number of factors, such as dam type, reservoir geometry, quantum of water to be diverted, topology, submergence, geotechnical conditions, environmental considerations, especially those related to fish life-sediment exclusion and ice formation, where necessary. Hence, this has to be decided on case to case basis keeping in mind the following primary considerations:

- Adequate inflow
- Least silt intake
- Least head loss
- Least negative environmental impact

Following guidelines or criteria should be observed to achieve the aforesaid objectives:

1. Intake approach should be symmetrical to avoid vorticity and to minimise entry head loss;
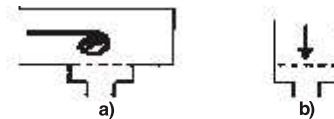
2. Submergence depth should be more than 70 percent  of the intake pipe diameter to avoid entry of air pockets and vorticity;
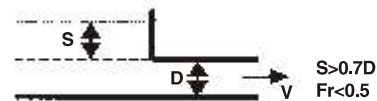
3. Off take from straight reach to be avoided, better coerce water to follow curved path to avoid larger entry of silts;

4. Off take should preferably be located on concave side near the end of the curved stretch to avoid larger entry of silts;
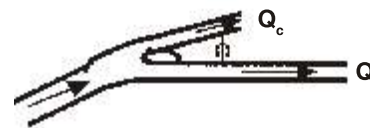
5. Angle of diversion should be $20°$ to $30°$ , if discharge ratio =0.2 to 0.3 & $45°$ to $60°$ & if discharge ratio = 1.0 larger entry of silt to be avoided. :

6.  Smooth flow transition should be ensured to minimise entry head loss. A sudden acceleration or deceleration of the flow generates additional turbulence with flow separation and increases the head losses.

The orientation of the intake entrance to the flow is a crucial factor in minimizing debris accumulation on the trashrack, a source of future maintenance problems and plant stoppages. The best disposition of the intake is with the screen at right angles to the spillway so, that in flood seasons the flow entrains the debris over its crest. The intake should not be located in an area of still water, far from the spillway, because the eddy currents common in such waters will entrain and accumulate trash at the entrance. If for any reason the intake entrance should be parallel to the spillway, it is preferable to locate it close to the spillway, so the operator can push the trash away to be carried away by the spillway flow.

### 3.5.2.3 Components of Intake Structure

The main components of Intake Structure are;

1)  Trash rack
2)  Trash rack supporting structure
3)  Stop logs & control gates
4)  Anti-vortex arrangements
5)  Bell mouth & transition

These are explained below;

### (i)  Trash racks

Trash rack is a screen provided at the intake to prevent entry of floating debris like grass, leaves, trees, timber etc., into the water conductor system. Each screen consists of vertical trash bars welded to space bars consisting of flat/channel sections. The screens are assembled in small panels for easy handling for maintenance. The trash bars are generally of mild steel flats with rounded edges at both upstream and downstream for smooth flow. The spacing of trash bars depend upon the type of turbine, its dimension and the peripheral speed of the runner. Following criteria / guidelines should be kept in mind while designing trash racks;

❑ Trash bars should be so spaced that the net opening between them should be at least 5 mm less than the minimum opening between turbine runner blades.

❑ The spacing of the bars should be adjusted so that the ratio of forcing frequency to natural frequency of bar is less than 0.6.

❑ The trash rack should also be designed to withstand the effect of submerged jets in the case of pumped storage scheme.

❑ The design loads for trash racks are the dead weight of the assembly, the water pressure and dynamic pressure of the floating materials. An unbalanced pressure is also developed on account of partial or total clogging of the racks. Emil Mosonyi suggests a differential head of 1 to 2 m under normal conditions and 4 to 5 m under exceptional conditions. U.S.B.R. recommendations are that the racks are to be designed to fail at 12 m differential hydraulic head for deeply submerged intakes and where submergence is 6 m or less, the head is to be taken as 2/3$^{rd}$ of the maximum depth of submergence. CWC practice is to consider 6 m of differential head in design to take care of dynamic pressure of floating material and unbalanced pressure due to partial clogging of the racks. 50% clogging is assumed in determining hydraulic pressure acting on racks.

❑ The velocity of flow in front of the screen has to be of such a value as to minimise the loss of head. Further, higher velocity may cause vibration in trash rack structure and may lead to its failure. The velocity of flow through the rack may be about 0.75 m/sec, if manual cleaning is resorted to and 1.5 m/sec, if the cleaning is by racking machines.

❑ Trash racks are to be cleaned frequently. For small stations with depth of racks 4 to 5 m, and where the floating material is small, manual cleaning is possible. If the floating material is large and height of trash rack structure is more, mechanical cleaning machines should be deployed for cleaning.

❑ Trash racks should also be inspected at least once in a year to timely detect any damage to the racks.

II)    **Trash rack supporting structure**

This is a reinforced concrete structure of columns (piers) and beams (ribs)

on which the trash rack screens rest. The structure may be vertical or inclined with respect to the axis of the penstock joining the intake. The Following criteria / guidelines should be kept in mind while designing trash racks supporting structures;

❑ The design of the supporting structure is made while considering the loads transferred by the trash rack, dead load of structure, dead and live load of the operating platform / top slab. A differential water head of three to six meters is considered depending upon the cleaning efficiency of trash racks. The columns and beams in the flow direction are shaped to enhance smooth flow.

❑ The shape of trash rack structure may be adopted to meet the requirements of the layout of head works layout and head loss. For instance, for high dams with nearly vertical upstream face, semi-circular trash rack structure is usually preferred to provide the required trash rack area economically. For low dams or diversion structures, a straight trash rack is usually preferred. Care should be taken to prevent dead zones of water and uneven or irregular flow patterns in the tunnel, formation of dimples, dye core and air core vortices, water circulation and other flow irregularities during operation in pumping, turbine or combined modes under symmetrical and asymmetrical operation of unit.

❑ No part of the trash rack structure should fall within 80 per cent of the intake height, $h_e$, from the centre point of intake.

❑ For an uptight semicircular intake structure, the racks should be located on a semicircle in plan with a minimum radius of $1.1428\ b_e$, where, $b_e$ is the width of opening. For an inclined semicircular intake structure, the racks should be located on a semicircle or a plane perpendicular to the axis of the structure and satisfying the other criteria as for the upright structure. In plan the racks would be laid out on an ellipse, the semi-major axis of which should have a minimum value of $1.1428 b_e\ /Cos q$, where $q$ is the inclination of the trash rack axis to the vertical. The semi minor axis of the structure is parallel to the dam face and would have a value of $1.1428 b_e$. The trash rack screens should be inclined in a three dimensional plane with a bottom corner of the tower screens resting over the base footing.

❑ Suitable fillet should be provided below the lowest screens to plug the

gap and effectively support the weight of the trash rack over the entire base.

❏ For shaft intakes, the racks should be located at 0.8 $D_1$ from the centre of the bell mouth, where $D_1$ is the inlet diameter of the bell mouth.

❏ The approach apron should not be placed closer than 30 percent of the intake height $h_e$ from the lower edge of the intake orifice.

### (iii)    Stop logs and control gates

Stop logs and control gates are provided for regulation of flow into the water conductor system. Stop logs are used when the intake gate needs maintenance and repairs. Grooves for stop logs and gates are provided generally in the intake body or piers. Following criteria / guidelines should be kept in mind while designing Stop logs and control gates;

❏ The operating platforms of stop log and gates are kept at such a level that the equipments are approachable for operation under all conditions.

❏ The control gate may be installed at the entrance or after the bell mouth section. In the former type, the gate may be operated from the top of the dam and in the later case, generally, it is operated through a shaft or gate gallery provided in the body of the dam.

❏ An air vent downstream of intake gate should be provided to release air pockets trapped along the inflow water. The air vent should be so designed as to admit air at the rate the turbine is discharging water under full gate conditions.

The area of air vent may be fixed by the following formula

$F = Q \times \sqrt{(S \times (D/t)^{3/2})} / (7500000 \times c)$,
Where, F = area of air vent pipe in m$^2$,
Q =   Maximum air discharge through penstock. Discharge of air through penstock is taken as around 20% of penstock discharge.
S  =   Factor of safety against collapse of pipe (normally 3)
D  =   Diameter of penstock in m,
t   =   Thickness of penstock in m, and

c  =   Co-efficient of discharge through inlet (0.5 for ordinary type of intake valves and 0.7 for short air inlet pipes).

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### (iv) Anti vortex arrangements

Requirement of submergence depth is primarily an anti vortex arrangement. Where the required submergence depth is not possible or vorticity is anticipated, additional arrangement is done to prevent formation of vortex at the intake. They may consist of reinforced concrete vertical fins constructed parallel to each other, Dinorwic louvered type or perforated breast walls. The details of these arrangements are finalised through model studies.

For the design of perforated breast wall, anti-vortex louvers and vertical fins, one meter differential head may be adopted.

### (v) Bell Mouth and transitions

The entrance is shaped in the form of a bell mouth so as to have a smooth flow and reduce losses. As already mentioned, the intake may be inclined or kept vertical with respect to the dam axis.

### Shape of inlet

The shape of inlet should be such so as to ensure uniform acceleration of flow. The entrances of penstock and conduits entrances are designed to produce an acceleration similar to that found in a jet issuing from a sharp edged orifice. The surfaces are formed to natural contraction curve and the penstock or conduit is assumed to be of the size of the orifice jet at its maximum contraction.

The normal contraction of 40 percent (coefficient of contraction $C_c = 0.6$) is to be used in high and medium head installations, 30 percent ($C_c = 0.7$) for low head installations and 50 percent for ($C_c = 0.5$) for re-entrant type intake.

### Opening Area

The opening area at the inlet = (Penstock area / $C_c$ x Cos f),
where, f = angle of inclination of penstock centre line to horizontal

### Height and Width of Opening

The height, $h_e$ at entry is calculated from the distance above and below

the intersection of the penstock centre line with the face of the entrance

$h_1 = D [ (1.21 \tan^2 f + 0.0847)^{1/2} + 1/(2\cos f) - 1.1 \tan f ]$

$h_2 = D [ (0.791 / \cos f + 0.077 \tan f) ]$

Where, D is dia of penstock

The opening height, $h_e = h_1 + h_2$

The width of opening, $b_e = (Area / h_e)$

**Shape of Opening**

As already mentioned, the inlet should be streamlined to minimise the losses. The profile of the roof and floor should be approximately to that of a jet equal as from the horizontal slot. The profile is generally an ellipse given by the equation,

$x^2 / (1.1D)^2 + y^2 / (0.291 D)^2 = 1$

The profile of sides should be such that it should generally be followed by equation:

$x^2 / (0.55 b_e)^2 + y^2 / (0.2143 b_e)^2 = 1$

While providing side flaring, it may be ensured that the size of opening at entry does not create any structural problems with the size of dam block or structure. In case, the dam block or structure, in which the intake is to be accommodated, has restrictions, the dimensions of side flaring should be restricted to that extent.

**Transition**

Gates need rectangular section for efficient operation and pressure pipe or penstock need circular section for its hydraulically efficient design. Hence, transitions from rectangular section to a circular section conduit is needed for achieving both the objectives. Sometimes transition is also required, when the cross-sectional area of flow decreases or increases due to bifurcation or merger. The transition should be designed in accordance with the following requirements:

1) Transition or turns should be made about the centre line of mass flow and should be gradual.

2) The cross-sectional area through out transition from rectangular to circular section and *vice-versa* should remain the same so as not to cause any acceleration or deceleration of flow.

3) Side walls should not expand at a rate greater than 5° from the centre line of mass flow.

4) All slots or other necessary departures from the neat outline should normally be outside the transition zone.

**Centre Line of Intake**

The geometry of the approach to the power intake should be such that it can ensure economy and better hydraulic uniform flow condition. The flow lines should be parallel, having no return flow zone and having no stagnation. Velocity distribution in front of penstock should be uniform and there should not be any formation of vortices. Formation of vortices at the intake depends on a number of factors such as approach geometry, flow conditions, velocity at the intake, geometrical features of trash rack structure, relative submergence depth and withdrawal Froude number $F_r$, etc.

To prevent vortices, the centre line of intake should be so located as to ensure following submergence requirements, which has been developed by an evaluation of minimum design submergence at prototypes operating satisfactorily.

For large size intakes at power plants ($F_r = v / ÖgD$ < 1/3), especially at pumped storage system, a submergence depth, h = 1 to 1.5 times the intake height or diameter is recommended

For medium and small size installations (Fr > 1/3), especially at pump sumps, submergence requirements may be calculated using the formula;

$h / D = 0.5 + 2 F_r$

This recommendation is valid for intakes with proper approach flow conditions. With well controlled approach flow conditions, with a suitable

dimensioning and location of the intake relative to its surroundings and with use of anti vortex devices, submergence requirements may be reduced below the limits recommended above. However, recourse to hydraulic model studies may be taken to determine more accurate value depending on the specific parameters of the particular structure.

### 3.5.3 Desilting Chamber

Most of the rivers carry heavy sediment load either in suspension or as bed load. The suspended load, especially the sharp edged fine sand (quartz) transported by rivers in hilly terrain causes rapid wear of turbine runner blades / buckets due to abrasion. This abrasion tendency increases with the head. In course of time, this may result in shut down of units for considerable duration thereby, causing enormous loss of power and revenue. Therefore, it is necessary to provide necessary arrangements for exclusion of sediments from the water. Normally sediments larger than a particular size (usually 0.2 mm) are removed from water entering into turbines. In run-of-river and canal hydel projects, when silt load is very heavy, sediment exclusion is done by sediment excluders & ejectors, which form part of the head works in the river. Desilting chambers, also known as silting tanks, settling basins, sediment traps, decantation chambers are used for removing sediments larger than the required size, which enter into water conductor system.

### 3.5.3.1 Types of Desilting Chamber

Desilting chambers can be classified into various types such as;

1. Natural or artificial based on the mode of construction
2. Manual or mechanical or hydraulic removal of deposition, on the basis of the method of cleaning.
3. Continuous or intermittent, on the basis of mode of operation.
4. Open channel or closed conduit on the basis of type of flow.
5. Single or multiple unit, on the basis of configuration / layout.

### 3.5.3.2 Hydraulic design of Desilting Chamber

Following aspects are to be taken into consideration in the hydraulic design of desilting chambers:

## (i) Location and orientation

The desilling chamber should be located near the head works/intake in order to reduce and control sedimentation. Yet, care should be taken to see that it is not too close so as to cause unwanted turbulence in the downstream of the intake/head regulator. The required head for flushing may not be available in the immediate vicinity of the head works in the case of hydraulic flushing. The chamber is to be located in the reach, where at least a straight length of ten times the average width of the channel or diameter of the inlet tunnel is available on the upstream, to achieve satisfactory distribution of flow.

## (ii) Inlet arrangement

The flow area in the desilting chamber is increased to reduce velocity needed to induce settlement. The increase in area is achieved by suitable horizontal or vertical divergence. For wide chambers, an expansion ratio of flatter than 1:4 to 1:5 is to be adopted for obtaining satisfactory distribution of flow.

In case of deep chambers in the tunnels, bed slope has to be steeper than that of the slope provided for wide chamber to prevent deposition along the bed. In such cases, a bed slope of 1:2.5 to 1:3 may give satisfactory results.

## (iii) Grids and other flow distribution devices

Grids / screens or other flow equalizing devices are provided at the end of inlet transition to reduce the turbulence and inequalities in the flow distribution. Screens / grids break large eddies into small ones. Screens having openings up to 60 to 80 per cent of gross flow area at the location of screen may be used for initial design. When the intermittent flushing is adopted, the bottom level of the grid has to be above the depth of flow during the flushing.

## (iv) Size of basin

The velocity of flow in the basin is required to be reduced to induce settlement. The flow area, i.e. the width and the depth of the basin is to be designed for limiting the velocity given by the critical velocity concept or

to keep the shear stress below the critical tractive force for the size of the particle for which the basin is designed. Generally, a flow through velocity of 0.3 m/sec for removal of sediment coarser than 0.2 mm and 0.15 m/sec for removal of particles up to 0.1mm dia. is considered in the design.

### (v)  Fall velocity of particles:

The fall velocity of sediment particles is to be obtained by laboratory analysis of suspended sediment particles collected at site. The length of the desilting basin depends upon the horizontal distance travelled by the particle within the time needed for the particle to fall from the top layer of the flow to the bed of the desilting basin. For preliminary design, the estimation of fall velocity may be obtained from Sundry's curve.

### (vi)  Sediment removal functions:

The efficient settling of sediment in a desilting chamber depends upon the fall velocity, whose measurement tends to become difficult due to turbulence in the flow. However, based on diffusion and probability theory, several functions have been proposed by Lamble, Rouse, and Camp. These functions are justifiable as they  are not based on the assumption of uniform distribution of suspended sediment along the vertical.

### (vii)  Outlet arrangement :

Proper arrangements are to be made at the outlet for skimming of the relatively less sediment laden top layers of flow. The settling efficiency improves with provision of wider outlets having higher silt level. The center line of the outlet should coincide with the axis of the desilting chamber for uniform withdrawal of flow over the entire widths of the chamber. Narrow outlets or outlets located on the side, lead to a reduction in the effective length of the chamber.

### (viii)  Bed slope in the case of intermittent flushing :

For efficient flushing of the sediment, a velocity many times more than the forward velocity of flow, during settling is required to be generated in the entire chamber. A steeper bed slope is, therefore, required for conveyance of the flow with a small hydraulic depth.

**(ix)    Size of the flushing outlet in the case of intermittent flushing:**

The sill of the flushing outlet has to be flush with the bed of the desilting chamber at the downstream end for transporting sediment in the channel. The flushing outlet should have an over all width equal to the bed width of the basin at out let.

**(x)    Size and slope of the hoppers**

In case of continuous flushing system, the bed of the desilting chamber is divided into a number of hoppers. In wide chamber, more than one row of the hoppers may be necessary. The slope of the hoppers is required to be steeper than the angle of repose of the suspended sediment to allow the sediment to slip into the openings at the bottom connecting to the flushing conduits / pipes underneath. The width of the hopper is, thus, related to the depth of the hopper, size of the opening at bottom of hopper and bed width of the chamber. In the case of narrow desilting chambers, instead of individual rectangular hopper, a continuous hopper on the bottom side with sediment accumulation trench below is preferable. The spacing of the openings between the flushing trench and flushing conduit is decided in such a way that the top of the dunes formed between the successive openings would not protrude in the settling zone above.

**(xi)    Size of flushing conduit**

Generally velocities larger than 3 m/sec are provided in the flushing systems. The velocity should increase towards the downstream with addition of flow from the chamber to the flushing trench. Normally, 20 percent of the inflow discharge is used for flushing of the basin, from which the size of the flushing conduit can be decided.

**(xii)    The size and spacing of the openings, from the hopper bottom to flushing conduit**

The first opening from the desilting chamber to flushing conduit is required to be larger to allow for the higher rate of deposition and larger size of particles. The size of the first opening has to be adequate to pass about 20 to 30 % of the flushing discharge with a velocity of 3 m/sec. The size of the flushing conduit at the beginning should have the same area. The total area of the opening can be estimated for passing the remaining

discharge with a velocity of 3 m /sec. The size of the openings is progressively decreased towards downstream as concentration and size of the sediment settling goes on decreasing towards downstream.

For this purpose, however, the total number of openings are required to be estimated. From the observations made in the models, it is seen that the dunes of the deposition are formed in the flushing trench of desilting chamber. The base width of the dunes in the direction of flow is about 3 times the height of the dune. The height of the dune on the bed flushing trench is to be fixed in such a way that it should not obstruct the flow in the settling zone. Taking into consideration the permissible top level of the dunes and the bed level of the flushing trench in the basin, the spacing of the openings can be estimated. Due to the slope of the flushing trench, the permissible depth of the dune may increase progressively towards downstream, advantage of which can be taken for increasing the spacing either for reducing the number of openings or reducing the flushing discharge or combinations of both.

Smaller size materials settling near the outlet end form a reverse ramp at the upstream edge of the skimming weir. The last opening has therefore to be a little larger than the opening just on its upstream.

### (xiii)   Escape Channel / tunnel

The velocity in the escape channel has to be more or at least equal to the velocity in the flushing system at its outlet at the tail end of the desilting chamber. The hydraulic parameters such as width, depth and slope may be calculated on the basis of Manning's' formula with appropriate roughness corresponding to the bed forms and its adequacy verified for the desired rate of sediment transport for the course material using appropriate sediment transport formula adopting the guidelines given by ASCE.

In the case of escape tunnels, the adequacy of the size may be ascertained and the head loss calculated using the criteria given for the design of flushing conduit.

### (xiv)   Location of Flushing Outlet

In the case of the escape channel, the sill level in the escape channel should be such that it discharges freely in the river during floods also. If the slope of the flushing channel is flatter than the slope of the river, which

would generally be in the case of diversion works in hilly streams, the outfall may be shifted further down to satisfy the above requirement. In the case of tunnel, it may get submerged during the floods. However, it may be ascertained that the residual energy in the tunnel after allowing for the head loss is adequate for letting out the desired discharge in the river. In both the cases, the outfall should be located in the forward region of the flow along the bank or on the concave side of the bend for further efficient transport of the sediment in the river.

## (xv)   Model Studies

Stilling basins are designed based on broad guidelines, assumptions and experience. Verification of these assumptions and adequacy of the layout as well as other design aspects is, therefore, required to be assessed by conducting studies in physical hydraulic models. These studies are generally conducted in geometrically similar scale rigid bed models for open channel type desilting chamber. In the case of closed conduit type chamber, transparent Perspex material is used for convenience of fabrication and for visualization of the flow in the chamber.

**(XVI)**  In run of the river schemes, provision for removal of suspended silt from water entering the water conductor system is often made  so as to avoid damage to turbine.  Example projects are Nathpa Jhakri, Chukha, Dhauliganga, Teesta-V, Chamera –II and Rangit.  In these projects, dam height above riverbed is of the order of 50m.

In some other projects, when dam height (above river) is high, reservoir has a large length & depth and design discharge is high, the provision of desilting system has been dispensed with (eg Chamera –I & Subansiri). In case of the projects with power house on the toe of dam and where penstocks are taken directly from concrete dam, it is not feasible to provide desilting system (eg Salal & Kurichu). In such case, problem of silt is taken care either due to low head and /or by providing intake quite above the spillway.

Number of desilting basins and size of each desilting basins should be considered not only on hydraulic consideration but also on geo-technical construction and economic consideration as well. Too many large caverns in less favorable ground condition pose construction challenges and should therefore, be avoided.

### 3.5.4    Head Race Channel / Tunnel

A channel / tunnel, which carries the water to the penstock / pressure shaft, is called Head Race Channel / Tunnel. It is different from the penstock / pressure shaft only in the context of water pressure. Water pressure in Head Race Channel / Tunnel is similar to the inlet pressure, whereas water pressure in penstock / pressure shaft is more, due to further drop in elevation.



*Head Race Tunnel Under Construction.*

Head Race Channel can be open channel flow (as in the case of run-of-river or canal hydel projects) or pipe flow (as in the case of storage hydel projects). Its design, therefore, follows open channel flow or pipe flow concepts, as the case may be.

### 3.5.4.1 Open Channel Flow

In case of open channel flow, flow in the channels in general is in the rough turbulent zone and the Manning's equation can be applied

$Q = 1/n \times (A\ R^{2/3}\ S^{1/2})$

Where, n = Manning's coefficient, which in the case of artificial lined channels may be estimated with reasonable accuracy,

A = Cross-sectional area of flow
R = Hydraulic radius = A/P, where P = wetted perimeter, and
S = the hydraulic gradient, which normally is the bed slope.

Typical values of Manning's co-efficient for different types of lining is available in most of standard text books / manuals. For concrete lined channels, n is generally taken as 0.018.

**(i)    Hydraulically efficient cross-section**

It can be seen from the Manning's equation that for the same cross-sectional area A and channel slope S, the channel with a larger hydraulic radius R delivers a larger discharge. That means that for a given cross-sectional area, the section with the least wetted perimeter is the most efficient hydraulically as it results in higher discharge. Semicircular sections are consequently the most efficient. A semicircular section, however, unless built with prefabricated materials, is expensive to build and difficult to maintain. Hence, next to this the most efficient section is a trapezoidal section, the half hexagon, whose side slope is 1 v. 0.577 h. Actual dimensions have to include a certain freeboard (vertical distance between the designed water surface and the top of the channel bank) to prevent water level fluctuations over spilling the banks. Minimum freeboard for lined canals is about 10 cm, and for unlined canals, this should be about one third of the designed water depth with a minimum of fifteen centimeters.

**3.5.4.2 Pipe Flow**

In case of pipe flow in the channels is, in general, in the rough turbulent zone and the Darcy - Weisbach's equation can be applied

Head loss $h_f = f (L/D) \times (v^2/2g)$

where f = friction factor, is a dimensionless number,
L = the length of pipe in m,
D = the pipe diameter in m,
V = the average velocity in m/s, and

g = the gravitational acceleration (9.81 m/s$^2$).

Friction factor f can be found from Moody's diagram.

In underground tunnels, construction of circular tunnels is difficult. Hence, a modified horse shoe section is used.

**3.5.4.3** In planning, design and construction of headrace tunnel, following considerations are taken into account:

- Plan alignment considering the geology to be met and topographical consideration.

- No. of construction faces, length and position of adits with a view to optimise the construction schedule.

- Size of the tunnel decided on the basis of the velocities, head loss and economic consideration

Excavation of tunnels demand high level of skills and mobilization (in lieu of equipment, manpower and finance). Large size tunnels particularly in weak rock conditions (when met during excavation), demand more specilisation and preparedness that is seldom available in the country and many times result in large rockfalls.

Such situations of loose falls can be avoided when general contractor is supported by a more specialised agency suited to particularly handle such poor reaches of tunneling.Cycle time for drilling blasting and rock support in tunnel excavation should be as less as possible. The same is feasible with good understanding of rock, well trained blasting crew, support arrangement supplemented by well managed dewatering arrangements. Advance ground probing during excavation can be an important component of the tunnel excavation.  For a project with long tunnel with many faces available for construction, a well experienced and knowledgeable contractor should be able to complete heading and benching excavation quite fast.

Concrete lining (normally plain concrete lining, 30 cm thick) of tunnels is done by continuous pour using specialized form works. Contractor's concrete production facilities, concrete pumps and formwork should be well designed for an accelerated placement rate.

### 3.5.5   Surge Tank

In steady flows, where discharge is assumed to remain constant with time, the operating pressure at any point along a penstock is equivalent to the head of water above that point. If a sudden change of flow occurs, for instance when the plant operator, or the governor system, opens or closes the gates too rapidly, the sudden change in the water velocity can cause dangerous high and low pressures. This pressure wave is known as water hammer and its effects can be dramatic; the penstock can burst from overpressure or collapse, if the pressures are reduced below ambient. Although being transitory, the surge pressure induced by the water hammer phenomenon can be of a magnitude several times greater than the static pressure due to the head.

Surge Tank is provided in water conductor system primarily to reduce the surge pressure to be considered in the design of penstock / pressure shaft. This economizes the design of penstock / pressure shaft justifying the extra cost in the provision of Surge Tank. The provision of surge tank has the following advantages;

i)      The length of the column of water gets reduced by placing a free water surface close to the turbine

ii)     It acts as a pressure relief opening to absorb surplus kinetic energy.

iii)    It acts as a balancing reservoir to supply / store additional water during starting / closure of gates / valves.

### 3.5.5.1 Type of Surge Tanks:

Surge tanks are generally of following types;

i)      **Simple Surge Tank** – This has constant diameter throughout. The maximum variation of water level is contained within the tank.

ii)     **Restricted Orifice Surge Tank** – This surge tank has restricted orifice at the entry point. Orifice area is generally kept at 0.75 times area of the penstock.

iii)    **Differential Surge Tank** – This surge tank has differential cross-sectional area. This has larger cross-sectional area at bottom for larger

*Surge Shaft*

volume control & lower cross-sectional area at top for higher pressure control.

iv)      **Air Cushion Surge Tank** – In this surge tank, air is injected or released to regulate  the pressure difference.

v)      **Surge Tank with Expansion Chamber or Spilling Arrangement** – This surge tank has expansion chamber in the middle and / or spilling arrangement to spill over water in up surge.

Selection of a particular type of surge tank depends on case to case basis. Simple surge tanks are very common, though restricted orifice & differential type surge tanks prove economical, especially in cases where large surge pressure is expected.  Sometimes multiple (more than one) surge tanks are also provided.

**3.5.5.2 Hydraulic Design of Surge Tanks**

Hydraulic design of surge tank includes determination of its optimum height & diameter to nullify the surge effects in the water conductor system. Following criteria must be considered in the hydraulic design;

i) The surge tank should be located at a place such that positive and negative water hammer pressures are kept within acceptable limits. The usual position is at the junction of Head Race Channel / Pipe and Penstock / Pressure Shaft. However, a surge chamber in tail race channel may be required immediately after the power house as well, if a long tail water tunnel discharges under pressure to prevent separation of the water column in draft tube and / or tail race tunnel in case of rapid closure & to prevent excessive pressure in draft tube and / or tail race tunnel in case of rapid opening.

ii) Surges, i.e., water fluctuations must damp out and be contained within the tank under all conditions.

iii) Surge range should not be so large as to cause undesirable governor movements causing difficulty in picking up load.

iv) Maximum upsurge level should be calculated when whole station is running at full load, reservoir water level is at maximum (MWL) and friction & other losses are minimum.

v) Minimum downsurge level should be calculated when there is full load rejection & instantaneous acceptance, reservoir water level is at minimum (MDDL) and friction & other losses are maximum.

One of the important aspects in the design of surge tanks is that the mass oscillations in the surge tank should damp out. D. Thoma has established a formula to calculate the minimum cross sectional area $A_{th}$ of the surge tank in order to ensure damping out of mass oscillations. The cross sectional area of the surge tank should be greater than $A_{th}$.

The cross-sectional area $A_{th}$ (Thoma criterion) is given by

$A_{th} = L.A_t / h_f. H_o \times V_i^2 / 2g$

The cross sectional area of the surge tank $A_s$ is given by

$A_s = A_{th} [1\text{-}1.5(1\text{-}k)]$,

where $H_o$ = Net Head;

$h_f$ = total head loss;

$A_t$ = cross-sectional area of Head Race Tunnel / Pipe
L  = Length of Head Race Tunnel / Pipe
$V_i$ = velocity of flow in Head Race Tunnel / Pipe
g  =  acceleration due to gravity
k  = station power generation / grid power

Factor of safety is usually taken as 2.0 for simple surge tank & 1.6 for restricted orifice tank.

Maximum and minimum surge levels is determined as under;

Surge amplitude $Z^* = V_o \ddot{O}(L.A_t / g.A_s)$

Maximum Up Surge Level from steady state level,

$h_{up} = Z^* + 1/3\ h_f$  (assumes smooth lining, n = 0.012)

Maximum Down Surge Level from steady state level,

$h_{dn} = Z^* + 1/9\ h_f$  (assumes rough lining, n = 0.018)

In case of restricted orifice $h_f$ = head loss – orifice head loss $h_{or}$

$h_{or} = C_d \times V_o^2/2g$, where $C_d$ = 0.6 to 0.9 & $V_o$ = flow velocity in orifice

**3.5.5.3** Surge Shaft excavation is carried out either by full sinking or pilot sinking followed by slashing. Geology of the site decides the choice as well as the height of the shaft. Excavation of shaft is generally a specialized job and skill and capability of contractor to handle the excavation plays a very important role

Surge shafts are usually provided with reinforced concrete lining.

### 3.5.6  Penstock / Pressure Shaft

The penstock conveys water from the intake structure to the powerhouse and can take many configurations, depending upon the project layout. Where the powerhouse is an integral part of the dam, the penstock is simply a passage through the upstream portion of the dam. A canal, pipe / penstock, or tunnel is required, where the powerhouse is separated from the intake. A penstock may be several miles long at diversion-type

projects. Water may be conveyed most of the distance at an elevation close to the forebay elevation via an open canal or a low pressure pipe or tunnel (Head Race Channel / Tunnel). The remaining portion of the penstock, where most of the drop in elevation occurs, would be a pressurized tunnel or Penstock / pipe. Because the cost of a pressurized tunnel or pipe is much greater than that of a low pressure tunnel or pipe, it is usually desirable to minimize the length of the high pressure penstock. When the powerhouse is located adjacent to the dam but is not an integral part of the structure, water would be conveyed through or around the dam via a pressure tunnel. For multi-unit installations, it is often desirable to serve several units with a single penstock, and manifolds or bifurcation structures are provided to direct flow to individual units.

### 3.5.6.1 Types of Penstock

Depending upon the nature of embedment, penstocks may be classified as given below;

### (i)     Buried Penstocks

Penstocks, which are installed under trench or fill conditions, are called buried penstocks. These are supported continuously on the soil / rock / backfilled materials and this arrangement therefore, economises on the penstock costs. Further, it has in-built advantages of protection from outside temperature, animals, anti-social elements, landslides etc. But, these are inaccessible & costly and difficult to construct, maintain & repair. These become desirable in case of semi-underground & underground power house and when the topography demands so.

### (ii)     Exposed Penstocks

Penstocks, which are installed above the terrain and supported on saddles, piers & anchors, are called exposed penstocks. These are supported intermittently at some regular intervals and therefore, require self weight of the penstock to be considered in the design. Further, it has the dis-advantages of being venerable to the effects of outside temperature, animals, anti-social elements, landslides etc. However, these have the advantages of being accessible and relatively easier to construct, maintain & repair. These are generally required in case of surface power house.

*105 MW Loktak Power Station (Manipur) - Penstocks*

**(iii)    Partly buried and partly exposed Penstocks**

Depending upon the topography, some penstock may be partly buried and partly exposed.

**3.5.6.2 Design Considerations of Penstock**

The design of penstock involves selection of material, determination of economic diameter, required wall thickness and selection of appropriate methodology for installation, viz., method of jointing etc. In addition to these, one has to give due attention to the design of support system (saddles, ring girders, stiffeners, piers, anchors etc.) and the design of other associated components (bends, reducers, bifurcation, manholes etc.).

**(i)    Materials of Penstock**

Penstock constitutes a major expense in the power house. Hence, appropriate choice of material of the penstock is very important. The following factors have to be considered when deciding which material to use for a particular penstock:

❖    Ground conditions, such as topography, soil type, weather conditions, likelihood of structural damage,

❖ Design pressure,
❖ Method of jointing, unit weight and ease of installation etc.
❖ Design life and maintenance,
❖ Availability,
❖ Relative cost, etc.

The following materials can be considered for use as penstock pipes;

➢ High strength steel,
➢ Mild steel,
➢ Unplastified polyvinyl chloride (uPVC),
➢ High density polyethylene (HDPE)
➢ Concrete, etc.

For large power houses, high strength steels are usually used. In case of medium to small projects, mild steel is used. In case of mini & micro projects, mild steel, concrete, uPVC, HDPE are usually used.

## (ii) Economic diameter

Since a penstock constitutes a major expense in a power house, determination of economic diameter of the pressure shaft / penstock is also very essential. A smaller diameter is less expensive, but has high head loss, which ultimately results in lower energy generation. Thus, a trade off is usually done between the annualized capital cost and annualized power loss due to increased friction loss. A more rigorous approach is to select several possible diameters and compute power and annual energy. The present value of this energy loss over the life of the plant is calculated and plotted for each diameter. The cost of the pipe for each diameter (with fittings etc.) is also calculated and plotted on the other side. Both curves are added graphically and the optimum diameter would be that closest to the theoretical minimum of the graph. Often empirical equations are available, which simplifies the above rigorous approach. A simple criterion for diameter selection is to limit the head loss to a certain percentage. A four percent loss in power is usually acceptable.

## (iii) Required wall thickness

Wall thickness of the penstock liner is designed to resist;

> The maximum internal hydraulic pressure with water hammer pressure, gravity loads & longitudinal stresses due to penstock movements,

> The maximum external pressure, when the pipe is empty & negative pressure as sometimes penstock can remain under the Energy Gradient Line

> To resist handling & transportation stresses

Maximum internal hydraulic pressure is pressure due to head difference between maximum water level & penstock center line level, minus the friction losses. Water hammer is variation in pressure due to changes in the rate of flow. The changes in flow rate can be caused by the turbine wicket gate motions due to power changes or load rejections, unit runaway and closure of the emergency valve or gate. The magnitude of the pressure variation is dependent upon the length of penstock, the velocity of the water and the rate of change of the flow. When the turbine gates close due to a decrease in load, the pressure increases above the steady full load gradient. As the gate movement ceases, the gradient drops below that for steady full load, then fluctuates with diminishing amplitude between the maximum and minimum positions until the movement is damped out by friction. When an increase in load causes the turbine gates to open, the gradient first drops below that for steady full load, then fluctuates in a manner similar to that described for gate closure. The penstock must be designed at every point to withstand both the maximum and minimum pressure at that point as determined by the highest and lowest position of the water-hammer pressure gradient.

The speed of pressure wave, i.e., wave velocity c is given by

$$c = [(K \times 10^{-3}) / \{(1 + (K.D. / (E.t))\}]^{0.5}$$

where K = bulk modulus of water $2.2 \times 10^9$ N/m$^2$

D = internal pipe diameter (m)

E = modulus of elasticity of pipe material (N/m$^2$)

t = wall thickness (mm)

The time taken for the pressure wave to reach the valve on its return, after sudden closure is known as the critical time

$T_C = 2L/c$, where L is the length of penstock.

For instantaneous closure, the pressure wave reaches the valve immediately after its closure, the increase in pressure, in metres of water column, due to the pressure wave is

$P = c. D_V / g$     where, $D_V$ is the velocity change

The water hammer pressure in steel pipes are more than three times greater than in PVC pipes due to the greater stiffness of the steel. There may not be any water hammer pressure, if closure time $T > 10\,T_C$.

If closure time $T_C > T < 10\,T_C$ then the reflected negative wave arriving at the valve will compensate for the pressure rise. Then the Allievi formula may compute the maximum water hammer pressure:

$$D_P = P_0 \left( \frac{N}{2} \pm \ddot{O}\frac{N^2}{4} + N \right)$$

where $P_0$ is the hydrostatic pressure due to the head and

$$N = \left( \frac{L V_0}{g P_0 t} \right)^2$$

where :     $V_0$ = water velocity in m/s
            L  = total penstock length (m)
            $P_0$ = gross hydrostatic pressure (m)
            t   = closing time (s)

The total pressure experienced by the penstock is $P = P_0 + D_P$

Once the water hammer pressure is calculated, required wall thickness can be determined from the following formula;

$$e = \frac{P_1 D}{2 s_f K_f} + e_s$$

where  e  = Wall thickness in mm
P$_1$  = Pressure in kN/mm$^2$
D  = Internal pipe diameter in mm
s$_f$  = Allowable tensile strength in kN/mm$^2$
e$_s$  = extra thickness to allow for corrosion / abrasion = 1.2mm
k$_f$  = joint or weld efficiency
= 1 for seamless pipes
= 0.9 for x-ray inspected welds
= 1.0 for x-ray inspected welds and stress relieved

Water hammer pressure has to be considered as positive or negative pressure and the penstock has to be designed for both the conditions. In addition to the negative water hammer pressure, negative pressure may also develop, when the penstock overt falls below the energy gradient line and this is given by the following formula;

Negative pressure P$_c$ (kN/mm$^2$) = 882500 x (e/D)$^3$
where  e = the wall thickness in mm, and
D = the diameter of the pipe in mm.

Allowable stresses to be considered in the design of steel penstock are;

for normal water hammer          = 60% of the yield strength and
= 38% of the ultimate tensile strength
for emergency water hammer      = 96% of the yield strength and
= 61% of the ultimate tensile strength.
Minimum thickness for handling and transportation
Minimum thickness = 0.25% of D + 1.2 mm. (as per ASME)
= (D+50)/400 cm where D is in cm (as per IS: 11639)

## (iv)  Penstock support & anchorage system

Exposed or non-embedded penstock require an extensive arrangement of support & anchorage system. This include the following

### Saddles
Saddles are provides for formed supporting the penstock and are called so due to its shape providing circular seating to the penstocks. These

provide continuous support and are designed for the full weight of the penstock with water.

## Support blocks or ring girders

Support blocks or ring girders provide intermittent support to the penstock. Its spacing is decided by designing the penstock as a continuous beam supported at these points. Generally the spacing is kept between 10 to 50 m.

## Anchor blocks

Anchor blocks are required, wherever there is a change in direction (vertical or horizontal) of penstock to resist the reaction forces caused by the change in direction of flow. These are designed as fixed support resisting reaction forces caused by the change in direction of flow.

## (v)　Penstock specials

Penstock specials are associated components of the penstock. These are;

## Bends

Bends (vertical, horizontal or combined) are necessitated due to topographical constraints. However, bends should be smoother and bend radius should be at least 5 times the penstock diameter.

## Transitions

Transitions (reducers or expanders) are necessitated due to change in cross-sectional area of the penstock due to merger, bifurcation / trifurcation or change in gradient. Convergence angle should not be more than $7°$ to avoid separation of flow.

## Bifurcation / Trifurcation

Often it becomes economical to have one Penstock / Header / Tail Race Channel & merge / split into two / three penstocks before entering the P. H. to feed water to each individual unit. This necessitates bifurcation / trifurcation, which need to be designed to withstand internal pressure

and properly streamlined to reduce hydraulic loss and cavitational damage. The layout and analysis become more intricate due to intersecting cones and the intersection of cones & cylinders.

**Manholes**

Manholes are necessitated to allow for periodic inspection of the penstock. Minimum one manhole should be provided. These have to be designed to have effective seal resisting internal pressure.

**Expansion joints**

One expansion joint should invariably be provided to accommodate expansion of penstock and relative movements of dam / power house.

**Sleeve coupling**

One sleeve coupling should be provided to accommodate vertical movement of dam / power house or foundation settlement.

**3.5.6.3** Pressure shafts are mainly vertical, but inclined pressure shafts have also been provided and constructed.  Inclined pressure shafts sometimes pose more construction difficulties in excavation and steel liner erection.

Depending upon the layout of the project, the height of pressure shafts varies from 200-300m to 600-700m range.  Diameters of the pressure shafts are of the order of 3-4m to 7-8m range.

When the project has more pressure shafts (2 or 3) and height of the pressure shaft is of the order of 200-300m, the raise boring or raise climbing is used to excavate pilot shaft followed by slashing.  However, when depth of pressure shaft is 100-150m range, it may be more economical to excavate by full sinking. Use of raise climber for moderate height of shaft and good rock condition also works well.

Very deep pressure shafts are fully steel lined. However, pressure shafts upto 150-200m height may be reinforced concrete lined, except the bottom horizontal portion, which may be steel lined.

Very deep shafts need special attention in the layout, excavation procedures, steel liner erection, construction difficulties, costs and

schedule. All these aspects need to be considered together in the design process and specialized agencies should be engaged during construction. Excavation of inclined pressure shafts of larger lengths can be carried out more accurately by use of inclined Tunnel Boring Machine (TBM).

### 3.5.7 Power House

Power house is the main area of a hydro power station. It is primarily an electro-mechanical field and civil structures are planned and designed to provide proper housing of electro-mechanical equipments. The function of various components of a P.H. for which hydel civil design should make adequate arrangements are given as under:

**(i)** **Spiral case and wicket gates** - to direct and control the water entering the turbine runner. The spiral case is a steel-lined conduit connected to the penstock or intake conduit, and it distributes flow uniformly into the turbine. Wicket gates are adjustable vanes that surround the turbine runner entrances and they control the area available for water to enter the turbine. Wicket gate settings are controlled by the governor.

**(ii)** **Turbine** - converts the potential energy of water into mechanical energy, which in turn drives the generator.

**(iii)** **Generator** - converts the mechanical power produced by the turbine into electrical power. The two major components of the generator are the rotor and stator, The rotor is the rotating assembly, which is attached by a connecting shaft to the turbine and the stator is the fixed portion of the generator.

**(iv)** **Governors** - regulates the speed and output of turbine-generator units by controlling the wicket gates to adjust water flow through the turbine.

**(v)** **Bus duct / Cables, circuit breakers, and disconnectors . Isolators** - link the generator to the power grid. Bus duct / Cables consist of the electrical conduits that transfer power output from the generator to the step-up transformers. Disconnectors / Isolators or circuit breakers are switches that connect and disconnect the generator to the power grid. Circuit breakers interrupt the circuit, when it is under load, and disconnects isolate equipment, once the load has been interrupted.

*Chamera Hydro Electric Project Stage-I, 540 MW (H.P.)*

**(vi)    Transformers** – are electrical devices that convert the generator output voltage into the voltage level of the transmission line.

**(vii)   Switchyard** – is a final link to the power grid. The line circuit breakers, disconnect switches etc. are located in the switchyard.

**(viii)  Control equipment** - necessary to facilitate the automatic or manual operation of various power plant equipment.

**(ix)    Auxiliary equipment** - consists of the electrical heating / cooling (air-conditioning) and ventilation, generator cooling, pipings, fire protection, overhead cranes and drainage systems.

**(x)     Erection bay** - an area provided for the assembly and disassembly of major generating components.

**(xi)    Service areas** - include offices, control room and testing rooms, storage rooms, maintenance shops, auxiliary equipment rooms and other areas for special uses.

**(xii) Draft tube** - conveys the water from the discharge side of the turbine to the tailrace. It is designed to minimize exit losses.

### 3.5.7.1 Design considerations for Power House

Followings are the general design consideration for any civil structure, which hold good for power house also;

➢ Safety against structural failure due to over stressing
➢ Safety against overturning
➢ Safety against sliding
➢ Safety against uplift pressure

Following **loads** need to be considered in the structural analysis of power house structure and its components;

**Dead load** - weight of the structure itself, including the walls, floors, partitions, roofs and all other permanent construction and fixed equipment.

**Live Equipment Load** – estimated load due to use of the structure. Equipment loads should take into account installation, erection, and maintenance conditions as well as impact and vibration after installation. This should include hydraulic & earth pressures.

**Wind Load** – This is important for design of overhead crane frames for surface power houses. However, this is not to be considered in case of underground power house.

**Seismic design Load** – This is due to seismic activity in the region and should be considered as per IS code. Design seismic parameters for major projects should be finalized through the National Committee of Seismic Design Paremeters.

**Construction Loads** – Loads which may arise due to construction activity and / or methodology are called construction loads. These are temporary loads, but often become critical.

**Stability Analysis** - should be made for each **monolith** concrete of the powerhouse and all **critical levels** should be investigated for the **most severe combination** of **horizontal** and **vertical forces**.

**Governing conditions –**

When powerhouse is separated from dam:

(a) Case **S-1**: head gates open—headwater at maximum level flood-control pool, hydraulic thrust, minimum tail water, spiral case full, draft tube full, uplift, and wind or earthquake.

(b) Case **S-2**: head gates open, tail water at powerhouse flooding level, spiral case full, draft tube full, uplift and wind or earthquake.

(c) Case **S-3**: head gates closed, tail water at draft tube flooding level, spiral case empty, draft tube empty, uplift and wind or earthquake.

(d) Case **S-4** (Construction): no tail water and no uplift.

When powerhouse and head works form part of dam.

(a) Case **M-1**: head gates closed, headwater at maximum level flood-control pool, minimum tail water (ice pressure (if applicable)), draft tube and spiral case open to tail water (uplift) and wind on upstream side or earthquake.

(b) Case **M-2**: head gates open, headwater at maximum flood level, tail water at powerhouse flooding level, spiral case full, draft tube full (uplift) and wind on upstream side or earthquake.

(c) Case **M-3**: head gates closed, headwater at top of flood-control pool, tail water at draft-tube flooding level, spiral case empty, draft tube empty, uplift and wind on upstream side or earthquake.

(d) Case **M-4** (Construction): no headwater, no tail water, no uplift and wind or earthquake.

**Notes** -

(1) In some cases, the maximum overturning moment will occur when tail water is at some intermediate level between minimum and maximum.

(2) In analyzing monoliths concrete containing draft tubes, the floor of the draft tube should not usually be considered as part of the active

base area since it is generally designed to take neither uplift nor foundation pressure.

(3) Monoliths should also be checked for lateral stability under applicable conditions and the possibility of their at high tail water levels should be borne in mind.

*Vertical force*- It is the sum of dead weight of the structure, weights of fixed equipment, supported weights of earth and water and uplift. The weights of movable equipment such as cranes and heavily loaded trucks should be included only where such loads will decrease the factor of safety against overturning.

*Horizontal forces* - due to headwater, tail water, ice, earth and wind or Earthquake pressures.

*Uplift assumptions* - If the powerhouse is separate from the dam, it should be assumed that uplift from tail water in 100 percent effective on the entire foundation area.

- If the powerhouse forms part of the dam, uplift assumptions should be the same as those for the dam.

- If the structure is founded on soil, uplift should be assumed to vary from headwater to tail water using the line of seepage method.

### 3.5.8 Tail Race Channel / Tunnel

The Channel / Tunnel, through which the water returns to the river after passing through the turbine is called Tail Race Channel / Tunnel. This is the last leg of the journey of water to watts. Its design as a channel / tunnel follows the same principles as in the case of Head Race Channel / Tunnel, except that often it has reverse slope. Another important criteria in design of Tail Race Channel / Tunnel is the determination of Tail Water Level, which actually determines the net available head, and therefore, power generation potential.

Maximum Tail Water Level is calculated by having design flood pass the river cross-section at its junction with the Tail Race Channel and finding out the depth of flow using Manning's equation. In major projects, this should be actually determined by carrying out hydrological observations

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(gauge & discharge) at the junction point. This maximum tail water level determines the turbine setting. This is fixed considering the following aspects :

It should be sufficiently low  to have high available head

It should be sufficiently high so as not to create cavitation (esp. in case of reaction turbine) & not to allow back flow from the river on downstream side in case of high floods.

## 3.6    GATES AND RELATED HYDRO-MECHANICAL EQUIPMENT

### 3.6.1   Introduction:

Hydraulic Gates and operating equipment thereof form important components of any water resource development project  involving dams, diversion works, outlets, tunnels, penstocks or hydropower plants. Their role is very crucial for successful operation of such projects.

Importance of equipment for gates in hydropower projects is well known. Their proper functioning is absolutely critical for the smooth running and intended   generation of envisaged hydropower.   Functioning of gates installed at a particular location of  a hydropower project like spillways, penstocks, tunnels and tunnel intakes draft tubes etc., to a large extent depends on  proper selection , design and operation .

Considerable changes in the approach to gates and related hydro-mechanical equipment  have been made over the past years. Though these perceptions were aimed at  improving the overall scenario as applicable to them, practices in this regard are continually being  improved so as to be in tune with the latest advances in other realms of the technologies  emerging lately . Due to radical changes in the practices, gates once considered popular are now obsolete. On the other hand, some types considered unsuitable for a particular application are found to be satisfactory  after being improved upon.

In following paragraphs, practices considered to be best and which are required to be adopted  for new age projects as well as for rehabilitating

and refurbishment of older generation installations are detailed. Description presented here is based on the experiences gained on the Indian projects as well as cases reported  worldwide.

### 3.6.2 Planning of gates and their operating equipment:

**Traditionally, any hydroelectric project will have gated installations at following locations:**

- Spillway of storage/diversion structure.
- Intake of  Water Conductor System
- De-silting  arrangements
- Surge shafts/ Tanks
- Inlet Valves
- Draft Tubes
- Tail  Race

Aspects related with the planning of  gates located at the above places should carefully incorporate features for the smooth and dependable functioning of the equipment. Some aspects to be looked into regarding planning associated with gates at above locations are as under.

### 3.6.3 Spillways

Spillways are equipped with such types of gates which are required to operate under high heads that depends on their sizes. Radial gates offer the best selection for large sized openings due to their in-built  features like absence of grooves which cause cavitation  damages and get clogged quite often and have high values of Co-efficient of discharge. Use of wheel mounted type of  vertical lift gates should be discouraged for sizes of spillways of dams bigger than 8 m width and 5 m high . In cases of low level diversion structures, wheel mounted type of  vertical lift gates for spans upto 18 m width and 4 to 5 m height have successfully been utilized on some Indian projects can be used as Hydraulic hoists  operating equipment in case of  large sized radial gates as they have distinct advantages as compared to the electro-mechanical rope drum hoists. Some of the positive aspects associated, with hydraulic hoists, are suitability for very high hoisting loads, smoothness of operation, damping of vibrations and reduced maintenance requirements. They are also suitable for remote/automatic operation with the help of sensors.

Electrically operated rope drum types of hoists are generally restricted to hoisting capacities upto about 125 t. in which case they offer affordable features like ease of the fabrication, economy and slender appearance. Stop logs are also provided on U/S of the radial gates for carrying out repairs of the gates as & when necessitated.

### 3.6.4 Intakes

Intakes are generally provided with suitably planned gates at the entry of water in the power tunnels/penstocks. Maintenance/guard gates as well as stoplogs for the bell mouth are necessary here. Some popular configurations and layouts are shown in 'fig.1'. In cases where turbines are not provided with main inlet valves, intake gates should be capable of closing within the specified time lest they attain 'runaway speeds'. Hydraulic hoists are essential features in such cases which enable fast closure of gates to suit a definite closure time. Orientation of gate seals and venting of gate shaft on the downstream are also important considerations. Specially in the case of high head gates operating under more than 15 m head, the hydraulic design of the gate is a very important consideration. Type of gate operation should be selected carefully based on these considerations. Where the gate is the key primary control, it is very much desirable to provide hydraulic hoist operations.

### 3.6.5 Desilting Arrangements

Gates at desilting arrangements require appropriate features suitable for exclusion of silt and control of discharges under high heads. These gates require sealing and bearing arrangements such that constant flow of silt laden water has no long term detrimental effects on the gate components. These gates are quite often of small size and are required to be operated under partial open conditions. In view of this, modern installations adopt hydraulically operated gates since they require positive thrust for maintaining partial operation. Gate bottom shape is also designed as in the case of regulating gates. Attempts in the past for providing electro-mechanical hoist and screw types of hoists have posed difficulties. A case in point is that of Chukha H.E. Project, Bhutan, where silt flushing gates were provided with a wire rope type of hoists. Due to heavy silt load these gates required frequent operation and rope drum hoists were found inconvenient. These gates could be made operable

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only after careful re-designing of bottom shape. In such installations attempts should be made to provide all the gated components of stainless steel as they require little maintenance during life time of the structure.

### 3.6.6 Surge Shaft Gates

Gates at the surge shaft location are essentially bulk heads and are operated under balanced water heads. Nevertheless, they are subjected to very high static column of water heads and require carefully designed embedment and gate bodies. Their operating equipment should be carefully planned such that while operating, they travel large heights with ease. Usually they are stored at the top of the surge shaft and are lowered only in case of power house shut down. Generally rope drum hoists are the only option for operating these types of gates but due to large rope lengths, specially designed guiding arrangements are necessary to navigate different layers of ropes wound on the drums.

### 3.6.7 Main Inlet Valves

Inlet valves are butterfly valves in most of the cases though the type of disc may vary depending on the water head. Best practices incorporate 'flow through' type of disc for limiting cavitation and vibrations for higher heads. They should be operated hydraulically with provision of pressure accumulators in case of power failure. For higher heads (usually exceeding 200 m), spherical valves are required to be provided due to their inherent resistance for water hammer forces.

### 3.6.8 Draft Tube / Tailrace Gates

Gates for draft tubes / tailrace are bulkheads type and are designed for the balanced operation. When these gates are lowered, it is normally expected that personnel are working in the power house pit for maintenance. In view of this, special care has to be taken for providing proper sealing arrangements. Older designs incorporated feature wherein accidental opening of intake gates when draft tube gates are lowered, would cause shearing off of screws of spring loaded attached guides thus causing gate to be dislodged from the sitting position and relieving the water pressure thus avoiding consequent permanent damages to the machinery of hydro power station. To safeguard against such eventuality these installations should incorporate an electrical interlock which will

prevent the accidental opening of intake gates when draft tube gates are lowered. Conventional arrangement of shearing screw can also be provided as additional safety feature. Further, draft tubes need to be equipped with water level sensors which will enable measurement of water level on the turbine side and indicate the balanced head position when the gates are to be lifted. Gates in the tail race are some times provided to isolate the reach of tail race channel between draft tube and its outfall. They are also bulk head types of gates and are meant to be lowered when the water level in the river is high.

While evolving lay out of gates and their operating equipment for all the above types of installations due regard has to be paid to the ease of maintenance, dimensions of the civil structure and their operational requirements. The causes of failures of several installations in the past have been directly or indirectly attributed to the faulty design or improper operation. Vibration of gates often poses serious problems endangering structural safety of gated installation. Failure to close fully is also a serious hazard which may lead to disastrous consequences. Though these issues form central issues for the basic designs of gated equipment, they are affected by planning of the layouts of hydro-mechanical equipment. In view of this, they should be properly addressed in the initial stage.

### 3.6.9   Guidelines for Design of Gates and Operating Equipment.

Design practices for gates are evolved after careful consideration of hydraulic and structural behaviour. Designers should attempt for evolving a fail safe design under all conditions of operation. At times this may prove to be quite uneconomical but importance of individual gates in a particular layout have to be considered and consequences of any failure thereof should be carefully foreseen.

Structural design practices have substantially stabilized now and detailed guidelines exist in Indian as well as foreign standards dealing with gates. Due to sustained contact of water as well as alternate drying and wetting, gate sections are prone to damages due to corrosion. Best practices should incorporate an inbuilt higher thickness of section for compensating such losses. Surface treatment and painting schedules should be evolved with due care. Usually gate surfaces should be painted with coal tar epoxy paints over suitable primer coats.

Gates are universally analysed on the basis of working stress method though modern techniques of analysis like Finite Element Methods have also been employed.

However, the more important aspect of design of gates is related with the hydraulics. In case of regulating gates hydraulics plays a crucial role. Modern practices of hydraulic design incorporate various improvised

## Table No.1

| Feature | Early Designs | Improved Modern Design | Comment |
|---|---|---|---|
| Bottom lip of gate | Flat or nearly flat | 45 deg. sloping | Improvement results in relatively low cavitation potential and less down pull |
| r/d=(radius of curvature/gate thickness) | Near Zero | 0.4 | Minimum cavitation |
| Gate slots | Wide | Narrow with slot width less than 2 inches | Narrow slots minimize flow disturbance at gate slots |
| Top seal and side seals | Effective only when gate is fully closed | Effective at all gate positions | Improvement prevents undesirable water jets at partial gate opening |
| Bottom seal | Babbit metal | Stainless steel | Resistance to cavitation |
| Material of gate lip | Cast steel | Stainless steel cladding and weld overlay | Improved resistance to cavitation |
| Gate profile | Curved in plan | Rectangular in plan | Smoother flow – minimum vibration potential |
| Slot profile | Rectangular | Downstream corners of gate slots offset with sloping transition | Smoother flow- minimum vibration potential |
| Hoist stem | Carbon steel, Chromium plated | Stainless steel or nickel copper alloy | Improvement eliminates corrosion due to porosity of chromium |
| Air vents | Inadequate | Adequate air vents based upon model studies | Minimizes cavitation and vibration |

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features which are evolved after careful consideration of flow-parameters and experiences gained in the past. As has been reported in ICOLD proceedings of New Delhi (1979 - Q.No.50) various features required for modern high head gates are reproduced in *Table No.1*.

### 3.6.10 Hydraulic Design Guidelines for Gates:

Based on past experience and model studies, certain design features are required to be incorporated in the design practices of gates to minimize hydraulic problems. The basic requirements of a good hydraulic design should be to achieve the following:

i)      The flow of water past various components of gate is streamlined as far as possible. If any eddies or vortices cannot be avoided, attempts are to be made to form them away from the vulnerable parts of the gate.

ii)     The pressures are positive to the extent possible. If negative pressures are unavoidable, their magnitude is kept small so that cavitation damage is avoided.

iii)    Gate vibrations are kept to a minimum.

### 3.6.11 Materials for Gates for Improved Functioning

Performance of gated equipment provided in the hydropower projects also depends to a considerable extent on the type of the material provided. Past installations have exhibited frequent bottlenecks in smooth operation due to flawed material selection. Modern installations adopt durable, strong and maintenance free materials so as to avoid malfunctioning and frequent breakdown. Improvement in the material science and increased availability of better material has provided considerable choice to the gate designers today. Some suggestions are as under:

i)      Provide stainless steel fasteners for seals due to their ease of replacement.

ii)     Adopt fluorocarbon (Teflon) clad seals for improved sealing performance and lower functional forces. They also have better life than ordinary rubber seals.

iii)    For wheel bearing and radial gate trunions provide self lubricating bearings with emergency greasing.

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iv)     Inaccessible gates can be made of stainless steel to avoid frequent maintenance.

v)      Encourage hydraulic hoists for the operation of various gates in power houses wherever possible.  They have an edge over conventional rope drum hoist in various respects.

## 3.7    GEO-MECHANICS AND INSTRUMENTATION

### 3.7.1   Introduction

Geo-mechanics has now matured as a complete science. Instrumentation is generally specified in each project plan and is recognized not only as a necessity but also a useful tool to the designers and construction and maintenance engineers for monitoring the safety of the structures.

Proper planning of instrumentation helps in gathering valuable information which enables to assess the performance and continuing assurance of the safety of the dam during construction, first filling of the reservoir and during long term operation of the dam.

Instrumentation as a tool for dam safety efforts could be one of the following three focus areas:

●     Meticulous planning by using computer modelling.

●     Maintaining safety through continuous rehabilitation and restoration.

●     Instrumentation for monitoring the dam during construction, first filling and during its  useful operation life

### 3.7.2   Need of Instrumentation

The instrumentation needs can be classified into the under mentioned general categories:

**Diagnostic**  Verification of design, verification of suitability of new construction techniques, diagnosing specific nature of adverse event etc.

**Predictive** For predicting behaviour of the structure

**Archival** To form a cumulative record of the behaviour of a structure  and

thus create a data bank that can be used in the future.

**Research**    Advancement of the state- of-the-art.

**Public**    To enhance public relation by assuring the continued safety of the structure.

### 3.7.3    Purpose of Geo-technical Instrumentation

**Site Investigation**

Instruments are used to characterize initial site conditions. Common parameters of interest in a site investigation are pore-water pressure, permeability of the soil, and slope stability.

**Design Verification**

Instruments are used to verify design assumptions and to check that performance is as predicted. Instrument data from the initial phase of a project may reveal the need (or the opportunity) to modify the design in later phases.

**Construction Control**

Instruments are used to monitor the effects of construction. Instrument data can help the engineer determine how fast construction can proceed without the risk of failure.

**Quality Control**

Instrumentation can be used both to enforce the quality of workmanship on a project and to document that work was carried out as per the specifications.

**Safety**

Instruments can provide early warning of impending failures, allowing time for safe evacuation of the area and time to implement remedial action. Safety monitoring requires quick retrieval, processing, and presentation of data, so that decisions can be made promptly.

**Legal Protection**

Instrument data can provide evidence for a legal defense of designers and contractors should owners of adjacent properties claim that construction has caused damage.

**Performance**

Instruments are used to monitor the in-service performance of a structure. For example, monitoring parameters such as leakage, pore-water pressure, and deformation can provide an indication of the performance of a dam. Monitoring loads on tiebacks or rock bolts and movements within a slope can provide an indication of the performance of a drainage system installed in a stabilized slope.

Ideally, instruments should have the following characteristics:

- Sufficient accuracy
- Long-term reliability
- Low maintenance requirements
- Compatibility with construction techniques and
- Low cost and Simplicity

### 3.7.4 Minimum Desirable Instrumentation

There is no simple rule which can determine the number of instruments, their exact type or location. Their determination remaining primarily a matter of experienced judgment. Though the actual cost of instruments may be relatively low as compared to the cost of structure itself but they can be very time consuming and expensive to install, measure and analyze data. It is, therefore, important to limit the number and type of instruments to be installed.

However following criteria can be adopted for minimum instrumentation in dams:

1) New dams should be designed to contain an appropriate number of instruments conforming to state-of-the-art and of desired quality and should be optimally located. This will help to assess the safety of dam on a long term basis consistent with engineering design and economic constraints.

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2)   The existing dams should be retrofitted with instrumentation as dictated by relative need on a site specific basis.

Minimum desirable instrumentation for concrete dam should be separately considered for small dams (say of height less than 30 m with no foundation complications) and large dams. The minimum type of instruments to be installed should be thermometers, strain meters, no-stress strain meters, uplift measuring pipes and seepage measuring device. Joint meters should be provided in concrete dams where, for structural reasons or otherwise, the contraction joints need to be permanently closed by grouting after the maximum contraction has been achieved. These are also provided where contraction joint movement needs to be observed to know the effect of temperature variation. For larger dams where deflection expected is appreciable, plumb line and triangulation method can be employed. The number of instruments deployed depends on the importance of dam and nature of foundation. Seismic instruments like strong motion accelerograph and structural response recorders are also installed in dams located in high seismic zones.

For smaller dams, unless there is foundation problem, for measuring the uplift pipes should be invariably provided. Seepage should be measured by 'V' Notches in gallery located at suitable locations. Displacement expected can be monitored by 'Triangulation'.

### 3.7.5   Parameters to be monitored

### 3.7.5.1 Masonry Dams

The dams are constructed as purely gravity structures relying on the self-weight to resist de stabilizing forces due to water. Hence, measurements of uplift and pore pressures in the body of the dam assume great importance. Due to predominantly manual construction methods, the permeability of the material varies widely and is generally greater than that of concrete. Hence, seepage has to be monitored closely in terms of quality and quantity especially it is desirable to carry out the the measurements of seepage for each block of the dam separately.

Since the material models for masonry can be applied in macro sense the importance of for displacement measurements by plumb lines or equivalent methods assume a vital role. These methods help in

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mathematical simulation of the dam behaviour and help in identifying the stress and strain concentrations in the dam.

The standard stress meters developed for mass concrete with aggregate sizes not exceeding 6" are not very successful in masonry. Similarly, the strain meters to be used for the masonry should have base lengths longer than those used for concrete dams. Longer gauge length strain meters are relatively difficult to install but should be preferred for masonry dams.

### 3.7.5.2 Concrete Dams

If a gravity or buttress dam maintains its structural integrity and is stable against sliding, no safety hazard is likely to occur. As contraction joints between blocks are much weaker than the mass concrete, any indication of loss of structural integrity in the dam or foundation will manifest itself at the joints. Instrumentation should be installed in all gravity or buttress dams to monitor relative movement between blocks exhibiting indications of previous movement or where movement might be reasonably anticipated. These measurements of relative movement between blocks should be correlated to other measurements that allow measurement of displacements of the dam relative to a remotely located fixed point.. Other significant parameters that should be monitored in the majority of gravity or buttress dams are uplift pressures and seepage including foundation drains. The results from these measurements may indicate open joints in the foundation, Additional monitoring devices should be installed to measure sliding or rotation along the plane. The quality of water should be determined for seepage, reservoir and tail water.

### 3.7.5.3 Concrete Arch Dams

Because of the monolithic behaviour of arch dams, displacement is the most meaningful parameter that can be readily monitored. Although displacements occur in all directions, the most significant displacements are the ones that take place in the horizontal plane. All concrete arch dams should have provisions for measuring these displacements, including relative movements between points within the dam and movement of the dam relative to a remote fixed point. In addition, the quality and location of all seepages, including formed and foundation drains, should be monitored whenever possible. Other parameters that should be considered for monitoring include foundation movement and

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relative movement at any major joint in the dam or in its foundation that is significant. The quality of water should be determined for seepage, reservoir and tail water early in the life of a dam to provide a basis of comparison with the samples which be collected subsequently.

Most spillway and/or outlet works structures experience some movement. Although movement in itself may not be a positive indication of structural distress, differential movement between structural elements or within different portions of the same structure usually is a definite indication. Means for determining differential horizontal and vertical movements should therefore be installed at vital locations on all existing structures. When measurable seepage is flowing from spillway and outlet drains or adjacent parts of a dam, quantities of flow and data for quality of water should be obtained.

The new dams should be designed not only to satisfy a minimum requirement but also to contain modern, high-quality, state-of-the-art instruments to be located at optimum positions and to the degree required to determine long time dam safety over a period consistent with engineering design and economic constraints. Existing dams should be examined and retrofitted with instrumentation as dictated by their relative need on a site-specific basis.

### 3.7.5.4 Typical Measurements

No general rule can be given for the type of measurements to be made at dams as they are of many kinds, have different site conditions and different problems. However the following  parameters  are recommended for measurements:

- Uplift Pressure
- Seepage
- Temperature
- Displacement
- Stress
- Strain
- Pore Pressure
- Seismicity

The purposes / reason for of monitoring the above parameters are briefly narrated as below

### i) Uplift Pressure

The hydrostatic pressure differential between the reservoir level and the downstream pool or tailwater results in an obvious potential for seepage through, around, and under a dam. Such seepage occurs at every dam through joints or cracks in the dam and through joints, cracks or bedding planes in the dam foundation and abutment rock.

It is important to measure these water pressures at various points in the dam foundation and abutments because such measurements can be critical in the detection of possible piping or other seepage induced instability situations, such as the presence of excess hydrostatic uplift pressures on the base of the dam. These measurements also indicate the gradient   (upstream to downstream) and provide the means to determine effectiveness of grout curtains and foundation drainage facilities.

Measurement of uplift in the foundation is mandatory for all gravity dams. It is important to determine the magnitude of any hydraulic pressure at the base of a dam. The effect of uplift on a dam is to reduce its effective weight on account of resulting buoyancy.

### ii) Seepage

Seepage is, undoubtedly, the best indicator of the overall performance of a dam because this reflects the performance of the entire dam and not just the condition at discrete instrumented points. Any sudden change in the quantity of seepage without apparent cause, such as a corresponding change in the reservoir level or a heavy rainfall, could indicate a serious seepage problem. Similarly, when the seepage water becomes cloudy or discoloured or contains increased quantities of sediment, or changes radically in chemical content, a likely serious seepage problem is indicated.

### iii) Temperature

Temperature is one of the fundamental parameter, denoting a physical condition of the structure. When the body of structure  is cooled, various primary effect can take place e.g. change in physical dimensions.

**Temperature during construction**

For concrete gravity dam it is very important to know the temperature variations in the dam during its construction which enables determining whether the concrete setting process is normal or otherwise. To achieve this, temperature measuring devices are embedded within the dam body and also mounted on the surface according to a predetermined plan for useful observations. Any abnormal setting process indicated by temperature observations may lead to a change in the accoumt of concrete lift and also changes in the treatment of aggregates before concreting and of the concrete during curing.

**Temperature of the dam interior**

It is necessary to measure temperature in the body of concrete and masonry dams in order to ascertain the nature and extent of thermal stresses and the consequent structural behaviour of the dam and also to ascertain when to undertake grouting of contraction joints that may have been provided for the structure

**Temperature of reservoir water and air**

Measurement of temperature of reservoir water and air is essential for distinguishing the effects of ambient and water temperatures on such measurements as deflection, stresses, strains, joint movements and settlements.

**iv)    Displacement**

Measurement of displacement of points either between two monoliths, or between foundation and body of the dam or the displacement of any joint of the dam with respect to the surrounding area will immediately reveal any distress conditions developing in the dam. Measurement of displacement is thus one of the most important factors to be studied while observing the structural behaviour of a dam.

**(i)    Internal joint movement**

Concrete and masonry dams are generally built in blocks separated by transverse joints. It is essential to know whether there is any relative movement between two blocks. The movement is likely to be due to

differential foundation behaviour. Further, the relative movement of blocks is also important from the point of view of grouting of transverse contraction joints.

### (ii)     Surface joint movement

Measurement of joint movement at the surface of the locations accessible from galleries is made by detachable gauges with a view to assess the amount of joint opening of two blocks of the dam. These gauges may also be advantageously used for observation of opening or of closing of surface cracks at any location.

### (iii)     Foundation displacement

Measurement of vertical or horizontal displacement of foundation provides information for taking measures for preventing inclination, distortion etc. of structures. The data can also be used for studying the elastic and inelastic properties of dam and foundation. Measurement of foundation displacement involves measurement of vertical and horizontal displacement for a  part of foundation with respect to dam.

### (iv)     Displacement of one part of the dam relative to other parts of the dam

Measurement of relative displacement of two points in a dam is direct indication of structural behaviour of the dam. The deflection characteristics of a dam observed for the first few years will reveal any dangerous tilt or movement of the dam. These observations are made by regular and inverted plumb lines. The plumb line data together with other supporting data may be used to study the elastic behaviour of the dam.

### (v)     Displacement of dam with reference to surrounding area

This measurement gives the absolute displacement of the dam with respect to surrounding area and is a direct indication of structural behaviour of the dam.

Provisions would be made for periodic deflection measurements. Where topography permits, this can be done by theodolite from fixed bases, using either line-of-sight over the top of the dam or by turning angles of targets on the downstream face and at the crest. For concrete dams, the

deflections should be consistent with changes in reservoir water surface level temperature and should not change appreciably from year to year.

**vi)     Stress**

Direct measurement of stress developed inside the mass of concrete or masonry helps in watching the structural behaviour of dams and their foundations. Any adverse change in stress will indicate distress conditions and remedial measures can be taken. The observation of stress also helps in studying the difference in assumed stresses and actual stresses in dams  and this can be used in improving upon the design procedure.

**vii)    Strain**

Factors like temperature, chemical action, moisture change and stress result in volume changes causing strain to the structure. The measurement of strain, therefore, becomes necessary. As the design of structures is based on stress it is essential to measure the stresses developed in the structures during their life time. But, the instruments available for measurement of



*Kohima Dam Pumped Storage Project (3x38 MW), Distt. Kamur*

stresses can measure only compressive stress and not the tensile stress. Further, the stress measuring instruments are more expensive and delicate than strain meters and hence, it is common practice to measure the strain and to calculate the developed stress from it .

**viii)  Pore Pressure**

Since large concrete and masonry dams are provided with internally formed drains internally & located near the upstream face, a record of pore pressure development and its variations would indicate the effectiveness and adequacy of these drains. Any sudden unusual increase in the pore pressures will be indicative of choking up of these internal drains and any unusual reduction from the normal would indicate possibility of formation of cracks or establishment of flow channels in the body of the dam.

Measurement of uplift in the foundation is mandatory for all gravity dams and is generally accomplished by providing uplift pressure pipes which provide a direct indication of the prevailing magnitude of uplift resulting from the operating reservoir heads and consequently the effectiveness of the grout curtain close to the upstream face of the dam and effectiveness of the drainage curtain provided in the foundation apart from checking of design assumptions for its stability.

**ix)  Seismicity**

A broad objectives of seismic instrumentation may be as follows :

a)  Seismotectonic Investigations
b)  Monitoring Fault Creep
c)  Earthquake Prediction
d)  Short Range Seismicity Studies
e)  Aftershock Sequence Studies
f)  Strong Motion Recording
g)  Measurement of Dynamic Properties of  Structures

**3.7.5.5 EMBANKMENT  DAM**

Earth dams differ from masonry and concrete dams due to relatively greater deformability and higher permeability of earth masses (excluding

plastic clay hearting). Strains and displacements in earth dams are therefore much bigger, hence comparatively simple instruments can be used for measurements of strains and displacements. Distribution of stress in earth dams is more complex and the design analysis is based on radical simplification of the stress pattern and shape of rupture planes. Consequently, stress measurements require considerable judgment in interpretation. Seepage is of greater significance as it can cause internal erosion as well as increase in pore pressure resulting in instability.

Embankment dams generally include earth fill or rock fill dams with impervious earth cores and the fill with upstream concrete or asphalt concrete facing including boulder pitching. The main purpose of instruments installed within the embankment dams is to study whether or not the dam is behaving according to design predictions. Requirements for measurement depend on the type of information desired.

There are some critical parameters which need to be measured for the purpose of monitoring. Generally, the similar type of instruments are suitable for earth and rockfill dams. For the latter, however, graded material is used between the rockfill and the instrument so that the instrument does not get damaged by the rock pieces.

## A. Typical   Measurements for Embankment Dams

No general rule can be given for the field measurement of  dams as site conditions and problems vary from dam to dam. Type of field measurements often needed to evaluate the performance of embankments are the reasons / purposes of monitoring are briefly narrated below:

1.    Pore Water pressure
2.    Seepage / drainage
3.    Deformation
        i)  Internal movement
        ii) Surface movement
4.       Seismic
5.    Reservoir and Tail water level
6.    Wave Height and evaporation.
7.    Rainfall
8.    Stress and strains

### 3.7.5.5   B. Pore Water Pressure

Excessive pore water pressures in either embankment or in the foundation or in abutments directly affect the stability of the dam. Such measurements can be critical because of possible piping or other seepage induced instability conditions, such as the presence of excess of hydrostatic uplift pressure resulting in loss of effective stress.

### 3.7.5.5   C.  Seepage / drainage

Seepage through, around or below an embankment dam is a valuable indicator of the continuing level of performance of a dam. The quantity of seepage entering a seepage collection system is normally directly related to the level of the water level in the reservoir. Any sudden change in the quantity of seepage collected without apparent cause, such as a corresponding change in the reservoir level or a heavy rainfall, could indicate a serious seepage problem. Similarly, when the seepage water becomes cloudy or discoloured, contains increased quantities of sediment, or changes radically in chemical content, is indication of serious seepage problem. Likely moisture or seepage appearing at new or unplanned locations on the downstream slope also indicate a seepage problem.

### 3.7.5.5   D.  Deformation

Movements conforming to normal expectations are basic requirements of a stable dam. An accurate measurement of internal and external movement is useful in controlling construction stability. The measurement of the plastic deformation of the upstream and downstream slopes under the various cycles of reservoir operation may indicate the likely development of shear failure at weak zones.

### i)  Internal movement

The measurement of internal movement of dams principally consist of vertical movements and relative horizontal movements caused mainly by the low shearing strength or the long term creep of the foundation or embankment materials. Internal movements do, of course, result in external movement of the dams crest or side slopes. In general, the need to measure vertical movement increases as dams increase in height and volume as this results in correspondingly greater

settlement than ,that for dams of lesser height on similar foundations. To provide data, that are readily interpreted, measurement of both the vertical and the horizontal components of movement at one or more locations may be necessary.

### ii) Surface movement

External vertical and horizontal movements are measured on the surface of embankments through the use of level and position surveys of reference points. Reference points may be monuments designated points on the crest, slopes or toe of the embankment or on appurtenant structures. Detecting surface evidence for problems of slope stability problems during construction is of primary importance. Such evidence includes slope bulging, sagging crests, foundation heaves at or beyond the toes and lateral spreading of foundation and embankments.

**3.7.5.5**    **E. Seismic effects**

Surveillance of seismic environment of the project site needs special attention in case of large dams to know about the seismicity of the region before taking up construction. Creation of a reservoir generates additional load on the surrounding area and underlying geological strata. Thus, it becomes essential to know the change in the seismicity pattern, if any, due to creation of large reservoir. The behaviour of dam during an earthquake also needs to be assessed.

**3.7.5.5**    **F. Reservoir and Tail Water Level**

Reservoir and tail water heads being one of the principal loading to which a structure is subjected, the measurement or reservoir and tail water levels is essential for interpretation and realistic assessment of the structural behaviour of the reservoir retaining structure.

**3.7.5.5**    **G. Wave Height**

Records of data for wave height along with wind velocity and other pertinent data help in deciding free board requirements more realistically.

**3.7.5.5**    **H. Rainfall**

This measurement is necessary for interpretation of pore water

pressure and seepage development in earth dams as well as to assess surface damage due to gully formation.

### 3.7.5.5    I. Stress and Strain

As the design analysis of earth and rockfill dams is based on radical simplifications of the stress, pattern and the shape of the rupture planes, strains and measurement of stresses, therefore, require considerable judgment in interpretation, hence strain/stress measurements may also be considered. Accurate measurement of stresses is difficult as distribution of stresses in earth and rockfill dams is complex  and is not  very successful except in case of bearing pressures of soil against rigid surfaces like wing walls and firm foundations. A unique relationship does not exist between stresses and strains in a fill dam. However, strains can be indirectly calculated using internal movement data.

### 3.7.6    BARRAGES

The diversion structures like barrage and weir are generally designed on the principles governing the percolation of water below the foundation of the structure. The floor of the structure is suitably designed either as a raft or gravity section to be safe against the uplift pressures created. Sufficient reinforcement is also provided    such that the permissible stress limits are not exceeded. Due to the various assumptions to provide for the 'unknowns' in the design, sometimes the design becomes a little conservative. Hence in order to know the 'health' of the structure under different loading conditions and also to know the progressive behaviour of the structure, there is  need to have instrumentation in the structure. By actual observations of their behaviour through instrumentation, the designs can be improved and economy effected. By having a continuous record of the observations with the instruments installed,  the distress spots in the structure can be located and  remedial measures to make the structure safe can be taken. Apart from this, the observations help in reducing the 'unknowns' and  place the future designs on sound  footing.

Measurements should be planned for the most important zones of structure. Planning of measurement should be made taking into

consideration the result of analytical and experimental investigation. Various types of measurements made on previously built similar structures can add in planning the necessary type of measurement for obtaining the needed information. The nature of measurement to be carried out on any structure will depend on the size and the importance of the structure.

### 3.7.6.1 Typical Measurements for Barrages

No general rule can be given for the type of measurements to be made at the dams as they have different site conditions and different problems. Type of field measurements often needed to evaluate the performance of embankments along with purpose of monitoring are briefly narrated below:

1. Water Level
2. Uplift Pressure
3. Displacement
4. Tilt
5. Stress and Strain
6. Vibration

### 3.7.6.1 A. Water Level

Measurement is useful for calculating the discharges passing over the barrage and for comparing the hydraulic jump condition theoretically expected with that of the jump behaviour actually observed. Water levels on the upstream of the barrage beyond the draw-down effect as well as on the downstream beyond the stilling basin need to be measured for correct computations. Water surface profile on either side of the dividing wall on left and right side are required to be made to assess the hydraulic jump condition

### 3.7.6.1 B. Uplift Pressure

Measurement is necessary to

i) Determine the actual uplift pressures occurring below the floor at different points and to locate the zones where the pressure is exceeding the safe balancing weight of the structure.

ii) Locate if any piping phenomenon is occurring anywhere below the floor.

iii) Ensure that the hydraulic gradient of the subsoil seepage flow is safe towards the end of the floor, so as not to exceed the safe exit gradient at the tail end of the barrage and

iv) Compare the theoretically computed uplift pressures with those actually observed.

### 3.7.6.1    C. Displacements

#### i)    Displacement of the main Barrage structure

It is sometimes observed that the adjacent units of the barrage structure separated by construction joints undergo relative displacements resulting in breaking of the copper seal at the joints. This is more common at the abutment toe where it is normally separated by expansion joints from the first or last bay of the barrage raft. Such displacement happens at the joint in the other bays of the raft as well. Once the seal is damaged considerable seepage can take place through the joint, resulting in reduction of the pond level. To take early precautions against any damage to the joint seals, relative displacements need to be monitored at the vulnerable joints

#### ii)    Displacement of the Flexible Protection Blocks

Flexible cement concrete blocks are constructed, cast in situ, just upstream and downstream of the pucca floor of the barrage or weir. Although these cement concrete blocks are normally meant to launch in case of scour in the river bed, the set of blocks immediately downstream of the pucca floor are not supposed to undergo any displacement as these are to release the uplift pressure at the exit end and at the same time prevent any scour or piping. However, in major barrages in alluvium, occasionally displacements have been noticed of these cement concrete blocks as well. This can never be seen when the hydraulic jump is actually forming over the stilling basin. As a result, considerable damage can happen to these blocks unnoticed under water. It is, therefore, desirable to monitor the displacements of these cement concrete blocks, at least in some

representative bays, by installing automatic checking devices fitted to these blocks. The measurement of displacement of the concrete blocks provided immediately upstream of concrete floor would also be necessary.

### 3.7.6.1    D.  Tilt

The abutment blocks, wing walls, divide walls etc. are tall isolated structures that may undergo tilt with or without vertical or horizontal displacements, particularly in seismic regions or where deep scour and differential pressures are expected on these structures.

### 3.7.6.1    E.  Stress and Strain

If the barrage floor is a relatively thin reinforced concrete raft constructed over alluvium, it is very difficult to calculate the exact stresses expected to develop in the raft concrete and the reinforcements. Theoretical calculations are generally done with simplified assumptions. It is, therefore, necessary to know the actual stresses and strains developing in the raft at least at the vulnerable locations.

It is also sometimes necessary to know the soil pressures under the barrage floor, piers or abutments particularly when the foundation is soft, weak or when differential settlement is apprehended.

### 3.7.6.1    F.  Vibrations

Barrages generally provide major rail and road bridges carrying heavy traffic. The induced vibrations may also necessitate monitoring especially for triggered stresses.

### 3.7.7    TUNNELS / UNDERGROUND CAVERNS

Hydro projects involve high risks and are very expensive Instrumentation of the underground openings is essential for their safety and cost optimisation of underground construction. In the absence of a knowledge about the rock mass behaviour, the excavation design would necessarily be conservative. An understanding of the rock stresses and deformation can result in potentially large cost savings in future construction.

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Analysis of stability is a principal geotechnical design task for underground excavations. Factors influencing stability include stratigraphy, ground water levels, strength of the soil or rock mass, geometry, excavation method, type of support and method of support installations.

Stability of underground excavations in the rock is usually controlled by the presence and orientation of discontinuities in the rock mass and the presence of water under pressure in these discontinuities. Failures most frequently occur as a result of sliding or separation along discontinuities.

Stability of soft ground tunnels in soft clay is usually controlled by the undrained shear strength of the soil in relation to the total overburden stress at the depth of the tunnel. In silts and fine sands, stability is usually controlled by the effectiveness of construction as well as de-

watering or compressed air in controlling the inflow of groundwater. In desiccated hard clays stability is usually controlled by the presence of fissures and slickensides.

Underground excavations for a project involve very often complicated and hazardous techniques, selection of which depends on many factors like location, alignment, cost, safety, construction efficiency etc. Many tunneling problems are caused by unexpected changes in the strength or deformability of the rock or soil mass where it is being excavated. When such a mass is disturbed, it undergoes a re-distribution of stresses, accompanied by a deformation / change of shape very often. These changes can be either inconsequential or catastrophic, depending on the distribution of stresses in the mass, its strength, deformability etc.          The changes reflected in geological structures may be rating to displacements, stresses, strains and pressures which can be measured using appropriate instruments.

Early detection of such changes can be of great importance, not only in identification of potential hazardous zones but in devising remedial measures and confirming their effectiveness.

The following parameters are generally required to be measured in the field to know the behaviour of underground openings and the support system.

### 3.7.7.1    Typical    Measurements Tunnels / Underground Caverns

No general rules can be given for the type of measurements to be made at dams as they have different site conditions and different problems. Type of field measurements often needed to evaluate the performance of embankments along with purpose of monitoring are briefly narrated below:

1.  Displacements/ Deformation
2.  Load
3.  Stress/Strain
4.  Pore Pressure
5.  Earth Pressure
6.  Seismicity

### 3.7.7.1 A. Displacement /Deformation

The measurements for displacement/deformation are required for obtaining the information on change in profile of an underground opening, to study effectiveness of roof / wall support system, to predict potential roof or wall falls before they actually occur and to monitor the movements in slopes and foundations

### 3.7.7.1 B. Load

Hoop load on steel ribs is measured by load cells. The data is used to estimate the support pressure on the steel ribs. Load cells also provide average value of horizontal and vertical support pressures. Sometimes, local geology e.g. inclined joints, require information on support loading along the dip of the joints. In such case contact pressure cells are installed at the steel rib & rock interface to measure directional loading on the steel ribs.

Pressures on rock bolts is measured by rock bolt load cells to evaluate adequacy of rock bolt design.

### 3.7.7.1 C. Stress / Strain

For assessing the circumferential stress within steel, pre cast concrete liner, cast-in-place or shotcrete liner is normally required. Observation of stresses is preferred because the strain data can not be meaning fully be converted to stress.

### 3.7.7.1 D. Pore Pressure

Pore water can be very critical sometimes and may have adverse influence on an underground opening. It is necessary therefore to measure pore pressure around underground openings.

### 3.7.7.1 E. Earth Pressure

For measuring pressure on and within lining of underground excavation and monitoring of stress and obtaining the information of rock joints.

### 3.7.7.1 F. Seismicity

Assessment and sustenance of structures under seismic loads pose a major problem in design and operation. The large reservoirs pose

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additional loading on the existing geo-tectonic set up of the region and their interaction with the tectonic environment needs to be monitored continuously throughout the investigations till their to operational phase.

To ascertain the distribution of Earthquake Epicentres in the vicinity of the site, as per the recommendations, five observatories around the site are required. Out of these five:

(i)  At least one should be Main observatory

(ii) And other four should be subsidiary observatory

(The spacing of observatory shall not exceed 70 kms)

Instruments for Main observatory:

(a) One complete set of high magnification Electromagnetic seismograph.

(b) Wood Anderson Type Seismograph

(c) One Accelerograph (two horizontal and one vertical component)

(d) Structural Response Recorders (this is dynamic model of structure)

Instruments for subsidiary observatory:

(a) One set of high magnification electro-magnetic seismograph

(b) Wood-Anderson Type Seismograph

(c) Structural Response Recorder

### 3.7.8    TYPES OF INSTRUMENTS

Most geo-technical instruments consist of a transducer, a data acquisition system and a communication system between the two. Many geo-technical instruments are not required to survive longer than the construction period. However many are required to survive as long as practicable. Therefore, selection of transducer, a data acquisition system and a communication system between the two depends on or duration of intended use.

**3.7.8.1** Instruments can be classified into following main categories based on their working principles:

1. Mechanical
2. Hydraulic
3. Pneumatic
4. Electrical / Electronic
5. Optical and
6. Fibre Optic sensors

The mechanical, hydraulic and pneumatic type of instruments are simple, rugged, reliable, cheaper and easy to operate. Their main weakness is lower response and lesser accuracy. The electrical / electronic are highly sensitive and have high resolutions.

The electrical/electronic type of instruments include un-bonded resistance type, bonded strain gauge type and vibrating wire type. The un-bonded resistance type of instruments have long term stability but they suffer from zero drift, cable resistance variation, sensitivity to temperature changes, moisture, movements etc. Their long term reliability is questionable.

Hence vibrating wire instruments are now increasingly being used instrumentation. These instruments are reliable, sensitive, accurate, durable and can be used with modern data loggers and computers. Other advantages of vibrating wire instruments  are :

·    splicing of cable can be performed without any significant impact on the long    term performance;

·    cable can withstand stresses due to construction activities; and

·    instruments do not need much maintenance.

**3.7.8.2    Factors Affecting the Choice of Instruments**

**3.7.8.2    A.  Critical Parameters**

Each project presents a unique set of critical parameters. The designer must identify those parameters and then select instruments to measure them.

- What information is required for the initial design?
- What information is required for evaluating performance during and after construction?

When the parameters have been identified, the specifications for instruments should include the required range, resolution, and precision of measurements.

### 3.7.8.2    B. Ground Conditions

Ground conditions often determine the choice of instrument. For example, a standpipe piezometer is a reliable indicator of pore-water pressure in soil with high permeability, but is much less reliable in soil with low permeability. A large volume of water must flow into the standpipe to indicate even a small change in pore-water pressure. In soils with low permeability, the flow of water into and out of the standpipe is too slow to provide a timely indication of pore-water pressure. A better choice in this case would be a diaphragm-type piezometer, which offers faster response since it is sensitive to much smaller changes in water volume.

### 3.7.8.2    C. Complementary Parameters & Redundant Measurements

The behavior of a soil or rock mass typically involves not one, but many parameters. In some cases, it may be sufficient to monitor only one parameter, but when the problem is more complex, it is useful to measure number of parameters and to look for correlation between the measurements. Thus it is common practice to choose instruments that provide complementary measurements.

For example, inclinometer data indicating increased rate of movement may be correlated with piezometer data that shows increased pore pressures. The load on a strut, calculated from strain gauge data, should correlate with convergence data provided by inclinometers behind a retaining structure. Another benefit of selecting instruments to monitor complementary parameters is that at least some data would always be available, even if one instrument fails.

### 3.7.8.2    D. Instrument Performance

Instrument performance is specified by range, resolution, accuracy,

and precision. The economical designer will specify minimum performance requirements, since the cost of an instrument increases with resolution, accuracy, and precision. Range is defined by the highest and lowest readings the instrument is expected to measure. The designer typically specifies the highest values required.

Resolution is the smallest change that can be displayed on a readout device. Resolution typically decreases as range increases. Sometimes the term "accuracy" is wrongly substituted for resolution. Resolution is usually many times better than accuracy and is never expressed as a "……." value.

Accuracy is the degree to which readings match an absolute value. Accuracy is expressed as a . value, such as . 1 mm, . 1 % of reading, or . 1 % of full scale.

Precision or repeatability is often more important than accuracy, since what is usually of interest is a change rather than an absolute value. Every time a reading is repeated, the value returned by the instrument is slightly different. Precision is expressed as a value representing how close repeated readings approach a mean reading.

## 3.7.8.2    E.  Cost-Effectiveness

The difference in cost between a high-quality instrument and a lesser-quality instrument is generally insignificant when compared to the total cost of installing and monitoring an instrument. For example, the cost of drilling and backfilling a borehole is typically 10 to 20 times greater than the cost of the piezometer that goes in it. It is false economy to install a cheaper, less reliable instrument. It is expensive and sometimes impossible to replace a failed instrument. Even when it is possible to replace the instrument, the original baseline data is no longer useful.

## 3.7.8.2    F.  Instrument Life

Are readings needed only during construction or will they be needed afterwards for years Instruments, signal cables, and protective

measures should be selected accordingly. Some instruments are excellent for short-term applications, but may exhibit excessive drift over the long term.

### 3.7.8.2 G. Environmental Conditions

Temperature and humidity also affect instrument choice. Instruments such as hydraulic piezometers and liquid settlement gauges have limited use in freezing weather. In tropical heat and humidity, simple mechanical devices may be more reliable than electrical instruments.

### 3.7.8.2 H. Personnel and Resources at the Site

The availability of personnel and resources at the site should be considered when choosing instruments. The following aspects also need to be considered:

· Do technicians have the skills required to install and read a particular type of instrument

Are adequate support facilities available for maintenance and calibration of the instrument?

### 3.7.8.2 I. Data Acquisition

An automatic data acquisition system may be required when:

(1) there is a need for real-time monitoring and automatic alarms;

(2) sensors are located at a remote site or in a location that prevents easy access;

(3) there are too many sensors for timely manual readings; or

(4) qualified technicians are not available.

If a data acquisition system is required, the choice of instruments should be narrowed to those that can be connected to the system easily and inexpensively.

**3.8    SAFETY ASSURANCE, MAINTENANCE AND REMEDIAL MEASURES FOR THE BEST PERFORMANCE OF HYDRO POWER PROJECTS.**

**3.8.1    Dam and its appurtenant works**

The most important aspect of Hydro Power Generation is to ensure that the plant continues to function safely. The main component of the Hydro Power Project is the dam, which may be of gravity, rockfill, Earth or combination of some of the above types. As these dams are very big in general, the safety of the dam is most vital for the unhindered performance of the power plant. Even a well- constructed dam would face many problems and difficulties, if it is not properly operated and maintained. Further, an unsafe dam constitutes a hazard to human life and property in the downstream reaches due to sudden release of artificially stored huge amount of water, in case of its failure. It, therefore, becomes an important perspective to see that such a destructive force is not unleashed on unsuspecting people. The safety of the dams and allied structures is thus an important aspect for ensuring public confidence in the continued accrual of benefits from the investments made and to protect the downstream area from any potential hazard.

**3.8.2    Dam Safety**

The primary responsibility of maintaining and assuring the safety of the dam lies with the *Owner* of the dam. In India, there are 4050 functional large dams and another 475 are under construction as on April, 2002 and most of the dams are owned by the State Governments or Government bodies. Realising the importance and nature of the task, the Government of India established a Dam Safety Organisation (DSO) in CWC in May, 1979 and, constituted a Standing Committee in August 1982 to review the existing practices and to evolve unified procedures of dam safety for all dams in India, under the chairmanship of the Chairman, Central Water Commission. Government of India, Ministry of Water Resources reconstituted the Standing Committee as the National Committee on Dam Safety (NCDS) in October1987. The Chairman, CWC is the Chairman of the Committee, while Chief Engineer, DSO, CWC is the Member Secretary and as on June, 2002 there are 28 members in the Committee comprising State Irrigation/

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Water Resources Departments, State Electricity Boards and undertakings/agencies/Boards of Govt. of India / States. The NCDS (a) monitors the follow-up action on the report on Dam Safety Procedures both at the Center and at the State level, (b) oversees dam Safety activities in various States and suggest improvements to bring dam safety practices in line with the latest state- of-art consistent with Indian conditions and (c) acts as a forum for exchange of views on techniques adopted for remedial measures to relieve distress

With a view to building up appropriate expertise at State level to cater to the requirements of the States, in pursuance of the Fifth Conference of State Ministers of Irrigation held in November, 1980, 12 States having significant number of dams have created State Dam Safety Organizations while other dam owning agencies have set up Dam Safety Cells within their organisations.

In order to have an effective Dam Safety Assurance Programme in the States, the Government of India in late '80s proposed Dam Safety Assurance and Rehabilitation Project (DSARP). Four States, namely, Madhya Pradesh, Rajasthan, Orissa and Tamil Nadu along with CWC, were included in the "Dam Safety Assurance and Rehabilitation Project" taken up with the World Bank assistance in 1991. The two primary objectives of the Project, were

1. *To strengthen the institutional framework for Dam Safety Assurance in CWC and in the four participating States; and*

2. *To upgrade the physical features in and around selected dams to enhance their safety status* as *required through remedial works, basic facilities additions, and flood forecasting systems.*

Under this project institutional set up of CWC at the Centre as well as in the four participating States has been strengthened through setting up of State Dam Safety Cells/ Organisations with adequate staffing, training of officers, purchase & installation of modern equipments etc. The guidelines on "Management of dam safety risks" prepared under the project, based on the latest concept of risk analysis in Dam Safety prepared under the project have been another significant achievement.

Basic dam safety facilities like providing access roads, back up power, instrumentation, installation of communication system, stockpiling of emergency material, etc., have been provided at 182 dams in the 4 States. Out of a proposed total of 55 dams, remedial measures were completed at 33 dams under this project. Thus, reservoir capacities have been restored to provide for assured irrigation/water supply/power generation which in turn will contribute to the economic development of the respective regions in the country. The four State Governments through their own funds are rehabilitating the balance 22 dams.

*Central Water Commission through NCDS has prepared and circulated a number of guidelines/Technical literature for use of dam engineers across the country on topics like data books, Inspections, Inspection formats and check lists, dam safety procedures, modes of failures and remedial measures, hydrology reviews, dam break studies and emergency action plans etc.*

### 3.8.3 Reasons for Dam Failure

Everything deteriorates with age and dams are no exception. With advancing age, the requirement of monitoring also enhances. Along with these, the lessons from past failures of dams are also to be carefully noted. Failures of dams have significantly contributed to the advancement of knowledge. On analysis of the data of past failures, it is revealed that many of the failures of dams have resulted due to overtaxing of spillway by unexpected flood (overtopping), deterioration of defective foundation and erosion of embankment material caused by inadequate control of seepage. These problems are rectifiable and can be taken care of provided these are properly investigated and detected in time. Sometimes, a compromise is made on structural dimensions for saving cost. In such cases, a careful balance has to be struck in reducing the risk to an acceptable level without raising the cost to a prohibitive level. Pace of failure is also an important factor for consideration. Failure of embankment dam is gradual and gives sufficient warning time where as failures of concrete dams are usually sudden and require immediate action.

Important modes and causes of Dam Failure are as indicated below:

| S1. No. | Failure | Causes |
|---|---|---|
| 1. | Foundation deterioration | a) Migration of solid and soluble materials; <br> b) Piping; <br> c) High pore pressures; <br> d) Under cutting |
| 2. | Foundation instability | a) Liquefaction; <br> b) Slides; <br> c) Subsidence. |
| 3. | Defective Spillways | a) Obstructions; <br> b) Inadequate Capacity; <br> c) Evidence of over-taxing of available capacity; |

| | | | |
|---|---|---|---|
| | | d) | Faulty gates and hoists |
| | | e) | Structural inadequacy. |
| 4. | Defective Outlets | a) | Obstructions; |
| | | b) | Silt accumulations; |
| | | c) | Faulty gates and hoists; |
| | | d) | Gate position and location. |
| 5. | Seepage | a) | Improper compaction; |
| | | b) | Differential settlement; |
| | | c) | Pervious embankment materials; |
| | | d) | Presence of roots stump and debris; |
| | | e) | Piping; |
| | | f) | Migration of solid and soluble materials |
| 6. | Operational mal functioning | a) | Over topping of dam due to improper reservoir operation; |
| | | b) | Slope failure due to excessive or rapid draw down; . |
| | | c) | Failure of mechanical equipment at the critical time due to improper operation maintenance. |
| 7. | Instability of dam | a) | High uplift, |
| | | b) | Over stressing; |
| | | c) | Faulty design and construction. |
| 8. | Defective or improper materials and their deterioration | a) | Alkali-aggregate reaction; |
| | | b) | Leaching; |
| | | c) | Poor strength of aggregate; |
| | | d) | Poor bonding; |
| | | e) | Weathering of concrete due to expansion/ contraction or Wetting/ drying; |
| | | f) | Damages due to cavitation and water hammer; |

|   |   |   |   |
|---|---|---|---|
|   |   | g) | Excessive velocity and vibrations |
| 9. | Reservoir margin defects | a) | Pervious ness; |
|   |   | b) | Instability; |
|   |   | c) | Inherent weaknesses of natural barriers. |
| 10 | Earthquakes or other natural calamities |   |   |
| 11. | Acts of war / sabotage |   |   |

### 3.8.4 Safety Assurance

Dam safety is a systematic approach to manage the dams so that benefits generated at great cost continue to accrue to the society. Dam safety assurance may be divided into four stages, namely inspections, investigations, design of remedial measures and execution of remedial works.

In many of the problems and the reasons for dam failures cases, careful investigations suggest remedial measures to be taken. However, in some cases, like geologic hazards, earthquake, seepage etc, the reasons are subtle and not susceptible to easy investigations. It is only during construction and operation that correct picture comes out. The key to understanding these subtle features lies in the surveillance.

### 3.8.4.1 Inspections

**i)      Bi- Annual Inspections**

Every dam needs to be inspected twice in a year i.e. before the commencement of monsoon and after its conclusion. These inspections are to be carried out generally by the engineers in charge of the dam in the formats circulated by NCDS with utmost care, so that every part of the structure is searched thoroughly for any signs of abnormality/ distress. The reports needs to be reviewed by the Dam Safety Organisation of the State/ Agency for assessment of further studies if any required.

## ii) Phase-I inspection

A detailed systematic visual inspection of those features relating to the stability and operational adequacy of the project components is required along with a review of pertinent, existing and available engineering data relative to the design, construction and operation of the dam and appurtenant structures. The inspection should also include test run of the communication, electrical and mechanical operating equipment and measurements are to be carried out by inspection & performance instruments and other devices. The inspection is to be carried out on each dam by a team of engineers experienced in the field of design, construction, maintenance, hydrology, geology and mechanical & electrical systems. The phase-I inspection will result in an assessment of the general condition of the structure with respect to safety of the project and conclude if additional studies, investigation and analysis are necessary and warranted. The report of the inspection is to be placed before the State Dam Safety Committee (SDSC) for prioritization of the remedial action if any required to be taken up.

As per the recommendations of the Dam Safety Procedures, the States shall arrange safety reviews of dams which are more than 15 m in height or which store 50,000 acre feet or more of water by an independent panel of experts once in ten years. However, in case of the projects funded by the World Bank, the bank insists a cycle of 5 years shall be maintained for carrying out Phase-I inspection for all the large dams in the state.

## iii) Phase -II investigations (Comprehensive Safety Review)

Based on the Phase-I inspection reports and the recommendation of the SDSC a special dam safety review should be carried out when there are significant changes in the condition of any dam, such as:

- Major modification to the original or design criteria.

- Discovery of an unusual condition at the dam or reservoir rim.

- After an extreme hydrological or seismic event.

- An increased consequence category of the dam due to any downstream or upstream development.

Generally the following areas may require investigations and reviews for the assessment of the quantum of remedial works needed for the rehabilitation of the dam and its appurtenant works by the expert agencies.

- Design Flood Review;
- Reservoir Routing Study and adequacy of spillway capacity;
- Seepage/Leakage problem .
- Free Board Adequacy;
- Assessment of Geology and Foundation Treatment;
- Performance of Spillway and Energy dissipation arrangements;
- Assessment of present status of Dam body Physical Characteristics;
- Seismological status of dam;
- Structural Stability Analysis of Dam;
- Spillway Gates and Irrigation Outlet Gates, Power Outlets and Operating Mechanism;
- Reservoir Sedimentation and Reservoir Rim Slope Stability;
- Dam Instrumentation and past structural performance of dam;
- Operational preparedness and emergency action plans.

The detailed report of the Inspections, investigations carried out should be placed before the State Dam Safety Committee/ Dam Safety Review Panel / Special Committee Constituted for suggesting suitable remedial measures.

### iv)   Design of remedial measures and their execution

Designs for remedial measures suggested by the experts as mentioned above should be taken up by the recognised agency/ organisation manned by qualified engineers with expertise in the respective technology and they should submit the design memoranda to the project authorities as per the agreed terms and conditions.

Responsibility of the execution of the rehabilitation measures lies with the project authorities and guidance from Dam Safety Review Panel/ Special

Committee Constituted may be obtained from time to time. After completing the remedial measures, a close monitoring of their performance shall be made and recorded for future use.

Safety assurance of the project and its components as per the procedures/ guidelines laid out will enhance its performance.

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